THE CONSTRUCTION

OF

ROADS AND PAVEMENTS

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BY

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FIFTH EDITION
NINTH IMPRESSION

McGRAW-HILL BOOK COMPANY, INC.
NEW YORK AND LONDON
1940

PRINTED IN THE UNITED STATES OF AMERICA

THE MAPLE PRESS COMPANY, YORK, PA.

PREFACE TO THE FIFTH EDITION

The revision of "The Construction of Roads and Pavements" was begun in 1937 with the expectation that the task might be completed by the end of the year. It soon became apparent, however, that nothing less than a complete rewriting of the major portion of the Fourth Edition would be acceptable, and the program of work was expanded accordingly.

In the manuscript as finally completed there has been included a new chapter dealing with soils engineering as it relates to highway construction; the chapters dealing with bituminous construction have been rearranged, and the text largely rewritten; the principles of road-surface stabilization have been discussed in detail; and much new subject matter has been introduced in the chapters on design, on concrete slabs, and on highway economics.

Throughout, an effort has been made to keep the text to a size suitable for a course for undergraduate students, and in consequence many of the principles are stated with only a minimum of discussion, in the belief that most teachers will have little difficulty in supplying illustrative problems to supplement the text. The descriptions of methods of testing have been eliminated, but references are supplied which will enable the reader to locate readily the official descriptions of the significant test methods.

Numerous references to researches and publications on highway engineering have been included in the hope that these will enable the advanced student to expand his program to any desired extent.

The author is indebted to a large number of teachers and professional associates for suggestions, illustrative material, and criticisms; to each of them he has expressed his appreciation. It seems appropriate to make more formal acknowledgment of the very helpful and cordial assistance of Anson Marston, H. J. Gilkey, Robley Winfrey, M. G. Spangler, W. M. Dunagan, F. E. Lightburn, and L. O. Stewart—all of the engineering staff at

Iowa State College. Special thanks are due Professor R. A. Moyer of the civil engineering staff at Ames, who has been a close and valued adviser throughout. However, the author alone must take the responsibility for errors and for defective reasoning on matters of theory or principle which may have found their way into the manuscript.

Thanks are also expressed for the faithful work of Esther Kreft and Ann Landon, who typed and retyped the manuscript, and to Tom Roberts, who prepared the drawings for the illustrations.

THOMAS R. AGG.

AMES, IOWA,
January, 1940.

PREFACE TO THE FIRST EDITION

"The Construction of Roads and Pavements" was written to meet the need for a concise presentation of approved practice in the construction of roads and pavements and of the principles involved.

The book is intended primarily for use as a text in a two- or three-hour course in roads and pavements, but numerous tables and typical designs and specifications have been included that should add to its value as a reference book for highway engineers.

The ultimate object of all highway engineering is the construction of durable and well-designed roadway surfaces. The attainment of this object involves selecting and testing the materials, assembling those that are suitable, and incorporating them in the roadway surface. A knowledge of the construction of roads and pavements is therefore the basis of highway engineering.

Much of the material used in the text has been prepared from notes that have been accumulating for several years. During that time the author has attended numerous conventions of highway engineers and has often discussed the various problems of roadway construction with engineers with whom he has been associated. Doubtless many of the ideas expressed in the text have thus unconsciously been absorbed. A considerable amount of material has been obtained from current periodical literature and acknowledgment has been given for material abstracted from the writings of other engineers.

The author is especially indebted to Mr. T. H. MacDonald, Chief Engineer of the Iowa Highway Commission, for his assistance and encouragement; to Mr. J. W. Eichinger, editor of the *Iowa Service Bulletin*, for valuable suggestions and especially for furnishing photographs for cuts; to the Illinois Highway Department for much valuable data; and to a score or more other highway engineers who have furnished plans, specifications, photographs or data on construction.

Special acknowledgment is made of the very valuable assistance of Mr. C. S. Nichols, assistant to Dean of Engineering, Iowa State College, who has assisted in the preparation and arrangement of the manuscript.

T. R. AGG.

August, 1916.

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THE CONSTRUCTION OF ROADS AND PAVEMENTS

CHAPTER I

ADMINISTRATION AND FINANCE

The construction of urban and rural highways in the United States is a governmental function and is therefore carried out in each political subdivision of the nation by an agency that is a part of the official administrative organization of that unit. The highway work of a state is delegated to a state highway department or the highway bureau of a department of public works. In the smaller political divisions, like the counties and the municipalities, highway administration is a function of the board that is elected to discharge all of the governmental functions of that subdivision, but usually with some particular officer, such as a superintendent or engineer, employed to deal with the technical phases of the highway program. The county highways are administered by the *board of county commissioners*, or *supervisors*, as they are designated. An exception to the general rule is found in a few instances where a county highway board is provided to deal exclusively with highway administration. In many states the "town," or township boards, also have jurisdiction over certain highways.

Employment of Highway Engineers.—Engineers secure employment in highway work by appointment to a particular administrative organization, such as that of the state, county, or municipality. This appointment is made in various ways by the responsible board. In some departments the appointment is based upon civil service eligibility or is made from lists of qualified persons prepared carefully by a personnel department. In other cases the appointment is made on the basis of a recommendation from the staff member under whom the new man will work, through political recommendation, personal friendship, or casual acquaintanceship. In many cases appointment as city

or county engineer is made on the basis of competitive bids for the position, a nefarious practice which is not uncommon in the smaller political units.

Civil Service in the Highway Field.—There has been continuing agitation over many years for the application of a merit system to the selection of employees of the state and municipal highway departments. Such a system is in effect for employees in the federal service (Public Roads Administration), several of the states, and a few of the larger municipalities, but this method of selection has made little progress in recent years. Those who advocate the establishment of a list of eligibles through a merit system contend that such a system has the following advantages:

1. The method insures the selection of the most competent available personnel for the work to be done.
2. The system insures that promotion and increases in compensation will be made in accordance with the record of service of those employed.
3. It stands as a barrier between the employee and the directing authority to prevent unjust dismissal and unfair salary adjustments.
4. The merit system removes the employee from subversive political influences and permits him to devote his entire energies to the work to which he is assigned. It relieves him from the demoralizing uncertainties that follow political changes in the department administration.

Those who do not favor employment by the merit system point out what they conceive to be its unfairness. The more significant objections are as follows:

1. There is no type of examination that can differentiate between degrees of efficiency of applicants, and unqualified persons will sometimes be secured through the civil service route.
2. It is held that under civil service superior authority does not receive the loyalty and consideration that would be forthcoming if appointment were due solely to selection by the superior officer.
3. It is contended that civil service tends to remove the incentive to individual effort to secure advancement and thus places a premium upon mediocrity.
4. It is found that the maintenance of discipline and high standards of performance is sometimes difficult where the staff is appointed under a merit system.

Opportunities in the Highway Field.—The foregoing will indicate the general conditions under which engineers are likely to obtain employment in the highway field and some of the problems that will confront the engineer in such employment. In the

face of the objectionable features of certain of the existing highway administrative systems, and the uncertainties of tenure in such service, it must be admitted that most of the municipal and state highway departments maintain organizations with high morale and of excellent technical ability. Service in this field has proved satisfactory to many thousands of highway engineers who have built for themselves positions of responsibility that are technically interesting and provide a satisfactory professional environment. It is to be expected that as governmental functions are scrutinized and stabilized in the long overdue effort to increase efficiency, the opportunities for employment in the highway field will be increasingly attractive.

Research Agencies.—It has long been recognized by those responsible for highway work that there must be a persistent effort to develop new facts with reference to the behavior of materials, the character of the soils upon which roadway surfaces are placed, the effects of the various practices followed in construction upon the vehicles that use the highways, and similar phases of the highway problem. The U.S. Public Roads Administration and most of the state highway departments have had continuously under way programs of research intended to develop facts that will be useful to highway builders. Many of the land-grant colleges also carry on a research program in fields related to highway construction. The results of these researches are disseminated through the publications of the U.S. Public Roads Administration, the Highway Research Board of the National Academy of Sciences, and the engineering experiment stations of the land-grant colleges.

ADMINISTRATION OF RURAL HIGHWAYS

The construction of highways being a function of government, some administrative device must be set up to insure that funds are provided, to allocate them to the several projects to be undertaken, and in general to do all those things required in prosecuting the improvement of highways. In each political subdivision of government there is some official organization that is charged with these responsibilities. The general nature of these administrative organizations will be outlined in sufficient detail to indicate how they function.

Federal Administration.—The federal government has developed an organization for highway administration because federal

aid[1] for highways is an established element of the system of financing the construction of trunk-line highways. Congressional appropriations for federal aid are made to the U.S. Federal Works Agency and allocated to the states on a basis set up

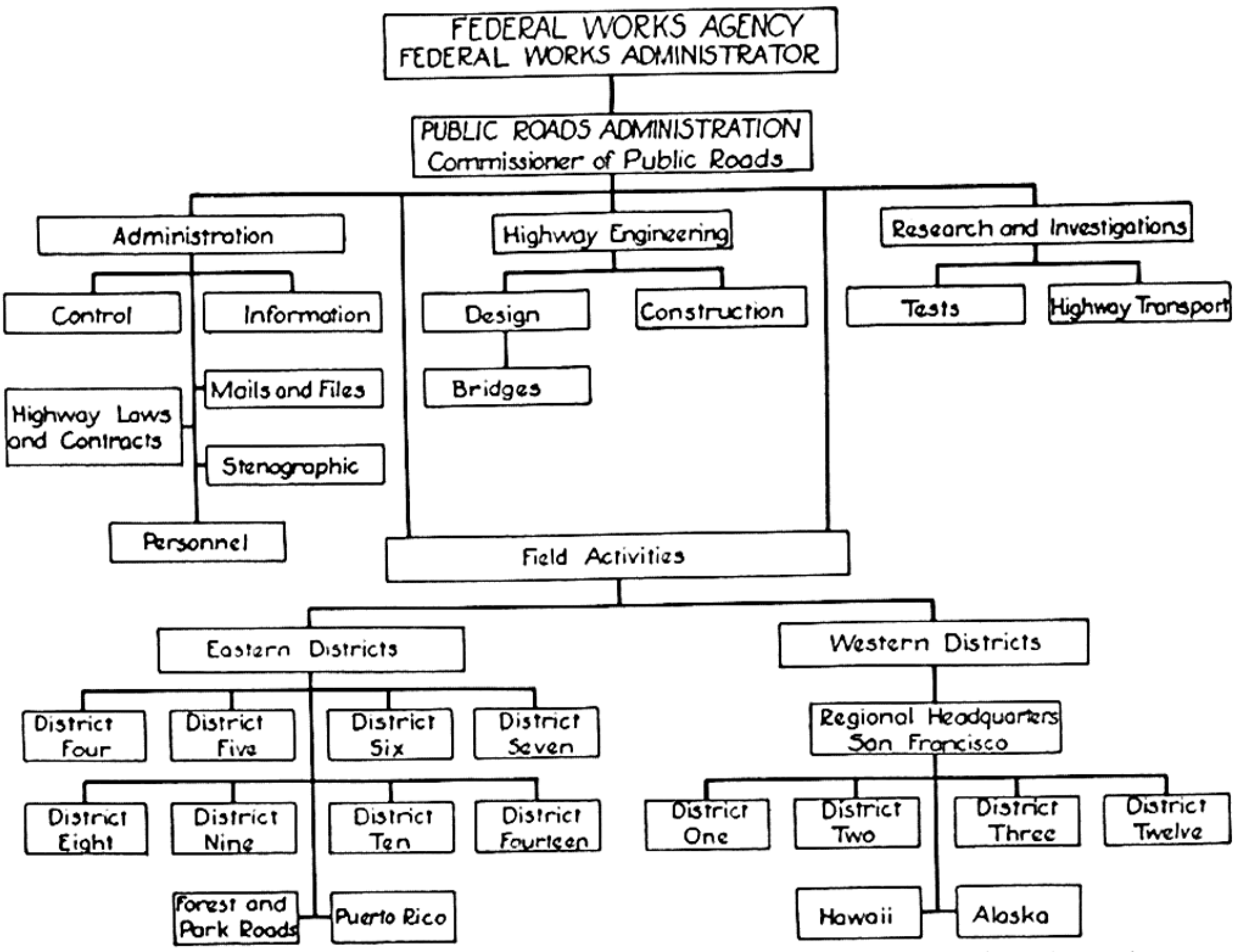

FIG. 1.—Showing the organization of the U. S. Public Roads Administration as of 1939.

by law. The responsibility for supervising the federal aid activities of the federal government is assigned to the Public Roads Administration which has the following administrative functions:

1. Direction of the allocation of federal aid funds to the various states, in accordance with law and subject to final approval of the Federal Works Administration.
2. The supervision of the expenditure of the federal aid funds allotted to each state.
3. The conduct of investigation on the construction and use of highways.
4. The gathering and dissemination of information with reference to all phases of highway construction and use.
5. The construction of roads in the national forests.

[1] LEWIS, E. A., "Laws Relating to Federal Aid for Roads," Superintendent of Documents, Washington, D. C., 1934.

"Regulations for Federal Aid," *Eng. News-Record*, Vol. 118, p. 278, Feb. 18, 1937.

The Public Roads Administration has a central office in Washington, D.C., and district offices at various places throughout the United States. All plans and specifications for federal aid highway work must be reviewed at these district offices and must be approved by the chief of the bureau before contracts are awarded. Likewise, the contracts for highway construction in which federal aid is involved must also be approved by the bureau. The staff of the bureau includes a large number of engineers, and these secure appointment from the civil service eligibility lists established through examinations that are held from time to time. The organization of the Public Roads Administration is illustrated in Fig. 1.

State Administration.—The administration of state highway work is carried out through a state highway department or a highway division of the state department of public works. At the head of these organizations is a board of three to five men who are appointed by the governor and who generally serve for fixed terms but may serve at the will of the governor. In general, there is a provision that a certain number of the members of such boards must be from each of the dominant political parties, and consequently such boards are of a political character.

The technical functions of the state highway departments are under the supervision of a state highway engineer or a state highway commissioner, who directs a staff of engineers and other specialists that carry out the work of the department. The state highway departments generally maintain a headquarters organization with a bureau chief for each of the principal technical activities of the department. These include such activities as design, construction, maintenance, materials testing, research, accounting, and similar activities.

The state highway departments have usually established division offices at appropriate places about the state, and each of these district offices supervises the construction and maintenance activities in its area. The district office may have a staff for design, for supervising construction, and for maintenance. Some state organizations also maintain district laboratories for the testing of construction materials.

The state highway departments of several of the states exercise some supervision over the work of county and township highway authorities, but there is no uniformity among the states as to the extent of such supervision. In general in such cases the state

has authority to approve the plans and specifications for county contract road work and perhaps also must approve the contracts awarded by the county boards. In certain cases the state highway department also exercises some control over the county highway budget. State highway departments in some states prepare standard or typical plans for bridges and culverts to

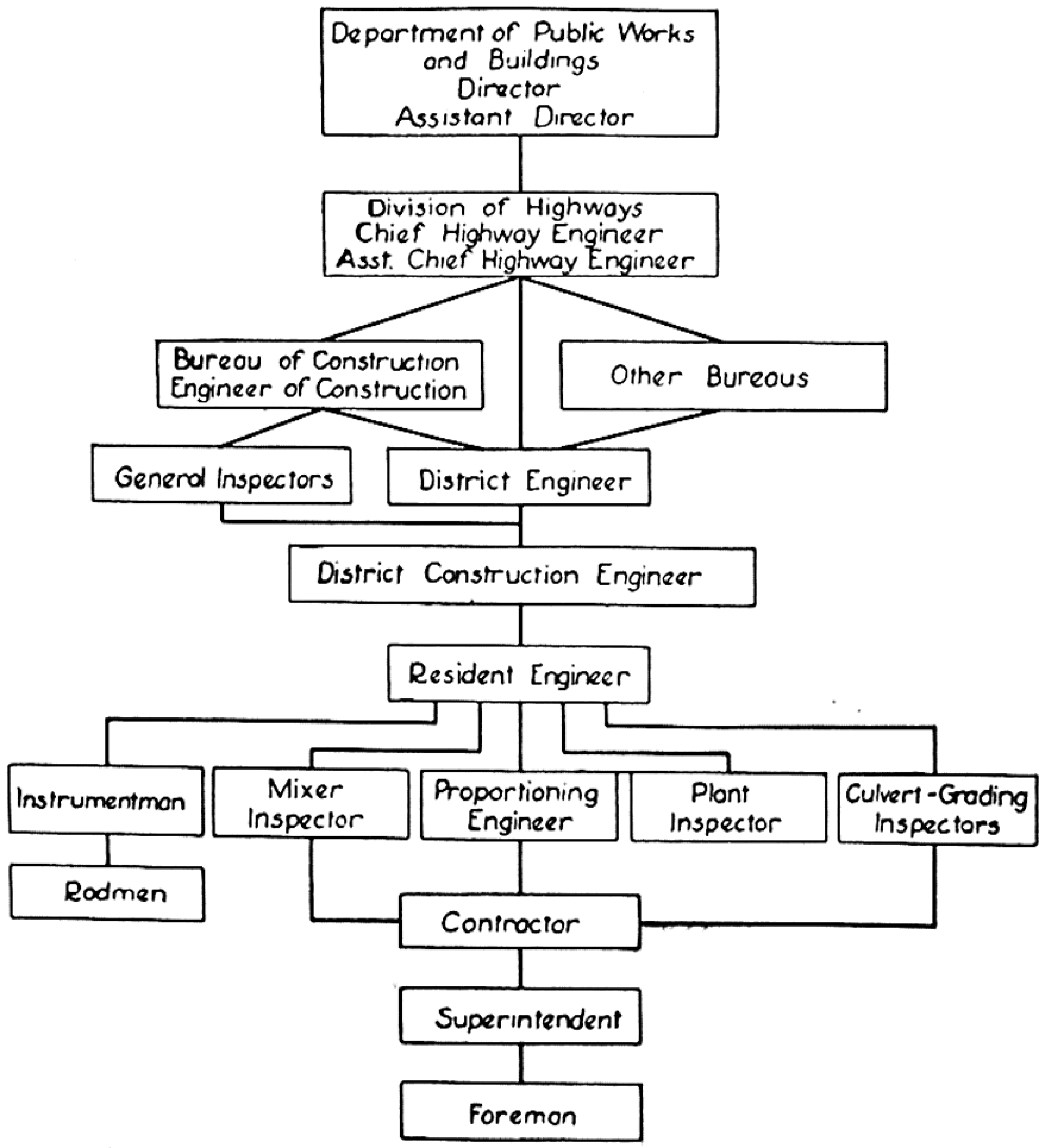

FIG. 2.—Showing the organization of the construction section of a state highway department.

which the county must conform. In other states the counties avail themselves of the technical assistance of the state, although they may not be obliged to do so.

It is rather the general rule for the state to build all the highway bridges (outside the cities) exceeding about 20 ft. in span.

In a few states the state highway department is responsible for the improvement of all of the roads of every class in the state, a plan that ought to spread, as it is logical and should be economical in the long run.

The principal function of the state highway departments at present is the supervision of the improvement and maintenance of the system of United States highways lying within the state and the state trunk-line highways. The organization of a state highway department is illustrated in Fig. 2.

County Administration.—After the federal and state highways had been laid out in each state, there remained a considerable mileage of roads the responsibility for the care of which was divided between the counties and the townships. In recent

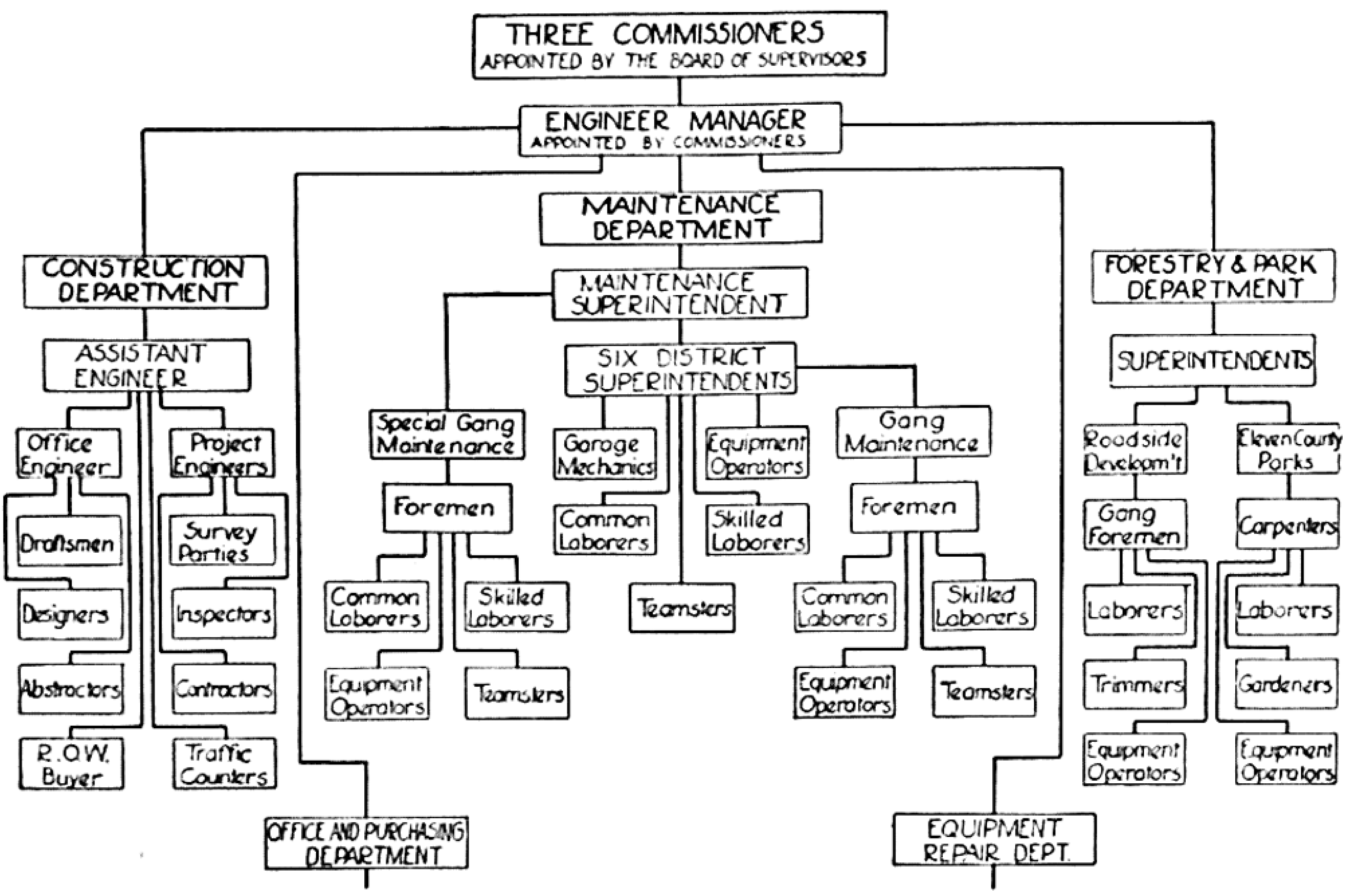

Fig. 3.—Showing the organization of a county highway department.

years the trend in the United States and in many other countries has been to place on the county, or its equivalent, the responsibility for all highways other than those included in the national and state systems, and this movement is likely to continue to grow until the townships fade out of the highway administration program.

The administration of county highway affairs is in the hands of the county board of supervisors or commissioners. These boards transact all sorts of business for the county, and the highway program is merely a part of their responsibility. They usually employ a county engineer to supervise the engineering work on the county highway systems, but it has been customary for the county boards to take the initiative in most matters

relating to this system and to make most of the decisions with reference to the type and amount of construction that should be undertaken. They do not hesitate to make decisions on technical matters, even though they may be entirely unfamiliar with what is involved. In a few of the counties in which large cities are located, county highway commissions have been established whose sole responsibility is the administration of the county highway system. The organization of a county highway department is shown in Fig. 3.

Township Administration.—For many years the townships ("towns") or subdistricts were responsible for the road work on the purely local roads. These boards are elective and have some responsibilities in addition to those connected with highway construction. Their principal duties are in connection with the maintenance of the local roads, and their compensation is usually on a per diem basis, so that they find it profitable to give considerable time to the actual work of maintaining these roads. The office of township commissioner often goes begging, and frequently those who are elected to the office are but poorly equipped for their responsibilities. The position has not been sufficiently attractive to men of ability; and as a result, the work has been inexpertly done and is quite inefficient. As mentioned before, several states (at this writing, Pennsylvania, Michigan, Iowa, North Carolina) have taken steps to do away with the township as a highway administrative unit, consolidating this portion of the highway system with that which is regularly administered by the county.

County Consolidation.—There are evidences that thoughtful persons consider it possible to reduce the cost of government and increase its efficiency by the consolidation of some of the smaller political units. One suggestion proposes, among other things, the consolidation of counties into districts made up of several of the present counties and the total elimination of township government. Such a consolidation would facilitate the prosecution of public works, because it would permit awarding contracts of a size that would be attractive to well-equipped contractors. It would also enhance the professional opportunity provided by the position of the engineer for the new unit, and he could be paid a better salary than is possible in a county. If, in addition to the highway work for the county, he also directed the work in the villages that are too small to justify

the employment of a resident municipal engineer, it would be advantageous to all concerned.

MUNICIPAL HIGHWAY ADMINISTRATION

The care of the streets in the cities and villages has always been a responsibility of the local government and discharged along with the other duties of the municipal authorities. Even in those cases where the incidence of through traffic has thrown a heavy burden on certain streets, the cities have long stood aloof from any cooperative relationships with the state or county highway authorities.

Municipal Administrative Authorities.—The administration of the improvement of streets and alleys of the municipalities is

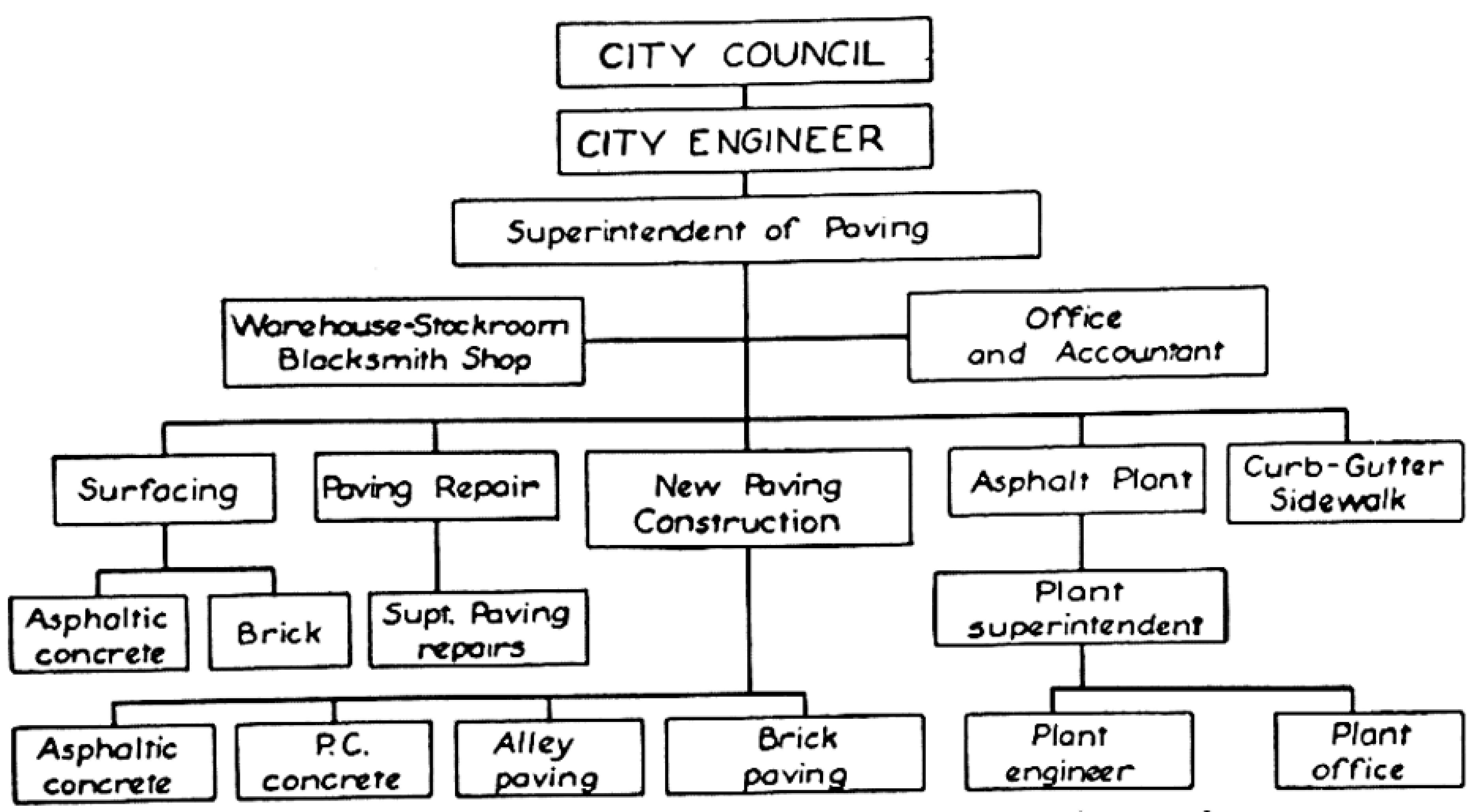

FIG. 4.—Showing the organization of a municipal highway department.

lodged with the city council or board of aldermen. If the city is of sufficient size to warrant the employment of a full-time engineer to supervise the highway work along with the other engineering functions of the city, that has customarily been the practice. In the large cities the volume of such work is so great that the municipalities maintain departments of public works to supervise all of the public construction in the city, and pavement construction is handled by a division of such a department. In the smaller cities it is customary to delegate the responsibility for the care of the streets to a committee of the board of aldermen, who function as a streets and alleys committee and carry on the routine work of the administration subject to the final approval of the city council. In all cases of improvement of

streets and alleys, plans and specifications for the work must be approved by the city council or board of aldermen, and likewise this board must authorize contracts in the name of the city when such improvements are to be made. The engineering staff employed for municipal highway work is usually employed by the city council and in most instances on a political basis. In a few cities, however, there are in effect civil service laws under which appointment is from a list of eligibles prepared through civil service examination. The small towns employ engineering firms for specific projects requiring engineering supervision.

In many of the larger cities the organization responsible for the maintenance of the streets is separate and distinct from the engineering bureau that supervises the construction of new improvements.

The weak link in the whole system of municipal administration has been that of inspection of construction. Even where the engineering organization has been established on the merit basis and is competent and fully manned, it is not uncommon to find that the inspectors on the construction jobs are appointed on a political basis. The results of this unbusinesslike practice can be seen in many cities, the character of the construction of the municipal highways being nowhere nearly of the degree of excellence found on the adjacent state highways. The organization of a municipal highway department is shown in Fig. 4.

Selection of Type of Street Improvement.—Selection of the type of improvement to place upon a street rests in the hands of the board of aldermen or city council. In some states there are provisions whereby the property owners along a street may petition for a certain type of improvement, and it becomes mandatory upon the city council to construct that type. In other states such petitions are to be considered suggestive only. If the residents along the street do not wish to have a street of the type decided upon by the city council, means are provided for making an effective protest against such construction. Although these various safeguards exist, it may be said that actually the selection of a type of improvement rests with the governing body. It is the rather common practice for city councils to take steps to determine which of the various types they consider satisfactory will be built at the lowest cost in a specific case. Sometimes this is done by taking competitive bids for several types that they

believe to be suitable for the work; in other cases they attempt to find which type is likely to be lowest in cost by securing preliminary estimates from the engineering department. On the whole the selection of the type of improvement for a city has not been very well handled. The engineering staffs, which possess the technical knowledge necessary for making a wise decision in these matters, have had comparatively little to say about the types of improvements. Political and commercial considerations have been powerful influences in many cases. It will be found that in many cities the condition of the streets shows the effect of a vacillating policy in these matters. In other cities some consistent policy has been followed throughout, and the financial benefit that results from an intelligent and enlightened public improvement policy is quite evident.

Federal Aid.—In view of the general benefits to the nation from the improvement of its highways, as set forth above, federal aid for highways is granted through Congressional appropriations. The aggregate appropriation for federal aid is allocated to the states on the basis of area, population, and mileage of rural delivery mail routes. Each state must match dollar for dollar the federal aid that is granted,[1] and all federal aid construction must be approved by representatives of the federal government. The justification for federal aid arises out of the general benefits of highway improvement and the fact that many industrial and commercial organizations profit from the convenience of improved highways but do not contribute directly to highway funds. They may not of themselves operate motor vehicles and consequently pay no license fees or fuel tax. In addition, they may operate enormous properties that do not pay any type of land or property tax to road funds. Such institutions usually do contribute to federal funds through the income and corporation taxes. Moreover, in many cases, the taxes are paid at the general offices which are in the metropolitan areas, whereas the properties are widely scattered throughout the country. It seems no more than fair that such enterprises should pay on a basis of the general benefits from road improvements, and the only way in which this can be accomplished is through federal aid. Federal aid seems to be firmly established in the program of the federal government

[1] This requirement was waived during the period following 1933 when various emergency plans were being tried to mitigate the effects of a serious "depression."

at the present time and has been instrumental in bringing about the development of what are known as the United States highways, which are those routes specifically employed for long distance travel.

Special Assessments.—Attempts have been made in the past to capitalize by special assessment against adjacent and abutting lands, the land access benefits accruing from rural highway improvement and projects have been financed in this manner, both in the United States and abroad. It is exceedingly difficult to determine the amount of such benefits in many cases; and if they could be determined, the amount of constribution to be required from the land would frequently be so small that the cost of administration and collection through the special assessment process would be relatively high. There are special instances in which the special assessment method would be advantageous and justifiable for rural highway improvement, but it is very little used in rural highway finance.

Excess Condemnation.—In view of the benefits that accrue to land made accessible by the improvement of highways, particularly in the vicinity of the metropolitan centers and in the cities, attempts have been made to secure such benefits for road funds. This can presumably be accomplished through the process known as *excess condemnation*. The method is to condemn and acquire for the public interest a tract of land lying along each side of the road being improved. When the improvement has been completed, the land so acquired, in excess of that necessary for rights-of-way, is again sold; and if there is increase in the value because of the improvements of the highways, this excess will be credited to road funds. This method has been tried out experimentally in a few places. It probably is justifiable in certain cases, but up to the present time there is some doubt as to its constitutionality in most states; in fact several states have amended their constitution to permit the use of this process. Highway engineers are likely to hear more about this method as time goes on.

In rearranging the street pattern in some cities in order better to accommodate traffic, it has been found that many small parcels of land are left adjacent to the new street lines. These small parcels cannot be developed advantageously and if privately held will be used for small and often unsightly structures, advertising kiosks, or billboards. If these parcels could be con-

demned and pass to municipal ownership, they could be developed to enhance the appearance of the street. The condemnation proceedings necessary are costly and time-consuming, and the allowances to the owners generally excessive, so the process has not been widely applied.

FINANCING STREET IMPROVEMENTS

The financing of street improvements presents difficulties not unlike those encountered in connection with the rural highway construction program, intensified by definite limitations imposed by law as to the methods of financing that may be employed. The present status of this problem can be set forth quite briefly.

Methods of Financing.—The method of financing street improvement, as originally worked out, consisted in assessing the cost of the improvement directly against the adjacent and abutting lands in accordance with the benefits that accrued from the improvements. A rather careful study of the effect of street improvement has shown that, in a very great percentage of the cases in which streets have been paved, the property has benefited in excess of the cost of the improvement. There have been some ill-advised expansions of the cities, and streets in certain unimproved areas have been paved with a view to making the property salable where the expected benefits were not realized. In the built-up portions of the cities, however, these benefits are generally admitted. Where elaborate boulevards or express highways have been built in the cities, the benefit to adjacent property does not usually warrant the assessment of the cost against such property.

As time passed and experience was gained with the special assessment system, it was found that in some cases, such as those mentioned above, the city was not justified in charging the whole cost of the improvement against the adjacent properties, and consequently the practice grew up of contributing a part of the cost of the improvement from general funds. This contribution was made in the form of payments for the cost of intersections, or payment of a portion of the cost of the whole enterprise.

For the more comprehensive projects of street improvements, such as the express highways mentioned above, a part of the cost is assessed against the abutting property, a part assessed against the property in a zone adjacent to the improvement, and a part paid from general taxes on all property in the city. Frequently,

bond issues are authorized for improvements of this character. In reality these through streets are really for the benefit of the traffic rather than to afford access to property, and that fact is the basis for the method of financing.

The problem of securing funds for the maintenance of municipal highways has become more and more acute as time has passed. It was originally the practice to pay the cost of such maintenance from funds derived from general taxation. In more recent years it has been found that such funds needed to be supplemented, and this has been done in some of the larger cities by requiring each vehicle owner to pay a municipal vehicle license fee. In the main the municipal vehicle license fee has not been well administered, has not been equitable, and has been most unpopular.

FINANCING RURAL HIGHWAYS CONSTRUCTION

Beginning about 1920, the traffic on the rural highways of the United States began to grow at such a tremendous rate that it became necessary for the states and the various subdivisions thereof to embark upon a quite ambitious construction program. These programs have continued to expand since that time, until by 1935 the annual expenditure for the maintenance and improvement of the highways of the United States amounted to several billions of dollars per year. The highway financing problems have been acute throughout this period, and in many instances taxation has been on the basis of expediency rather than on any systematic plan that tended to take account of the principles involved.[1] Nevertheless, several agencies have been active throughout this period in an attempt to stabilize the highway taxation program and to avoid the imposition of unjust taxes.

Some idea as to the nature of the benefits that accrue on account of rural highway improvement is necessary in order to develop a program of highway finance that is just to all interests. The more important of these principles will be enumerated with a view to emphasizing the desirability of providing for the highway program in such a manner that the taxation will be somewhat in accordance with the benefits received by the various interests that are taxed and at least approximately in accordance with ability to pay.

[1] AGG, T. R., "Taxes and Transfer of Control," *Eng. News-Record*, Vol. 117, p. 722. Nov. 19, 1936.

General Benefits.—There are certain benefits growing from road improvement that are so illusive and intangible that it is exceedingly difficult to say that any particular group of citizens is especially benefited. These benefits are beyond and in addition to the direct benefits that accrue to highway users; they include such things as the following:

1. Good roads affect the facility with which the children in the rural districts may avail themselves of educational opportunities.
2. Good highways make available certain agricultural products of a perishable character to the metropolitan areas, which would otherwise be deprived of them.
3. The general benefits include the social understanding that grows out of the wide acquaintanceship that people form through long distance travel.
4. A network of national highways is a distinct asset in the national defense.

All these must be classed as general benefits; and if road revenues are to be obtained from sources that reflect this general benefit, then these funds must be secured through a taxation system that reaches all classes of people.

Benefits to Traffic.—The most direct and tangible benefit from highway improvement is that which accrues to the traffic that uses the highways. This benefit exists in the form of lowered transportation costs and the increased comfort and speed with which travel can be accomplished. The benefits to traffic should really be divided into two elements, one of which might be designated as a readiness-to-serve benefit, and the other as a use benefit.

1. *Readiness-to-serve Benefit.*—The readiness-to-serve benefit exists because the highways have been constructed and are available for service whenever those who wish to travel care to avail themselves of this facility. This benefit is equally available to every citizen who possesses a vehicle with which he may go out on the highway.

2. *Use Benefit.*—The use benefit is that which accrues to the people who are actually using the highways, and it exists because of the possibility of traveling with comfort, safety, and speed at any season of the year. The magnitude of the benefit to any individual depends upon the amount of travel he finds necessary in the course of the conduct of his business and social affairs.

Land Access Benefit.—The land access benefit applies both inside and outside the cities and exists because of the increased

value that may accrue to land to which there is access from all-weather roads. With but relatively few exceptions it has been found that the value of urban properties increases when the streets have been paved, provided the improvement is limited to a design that is no more than reasonably adequate to serve the adjacent property. There may be instances in which farm lands are not benefited by the improvement of the highways that give access to these lands. Sometimes the lands are so poor that it is a matter of indifference whether they are readily accessible or not. In the main, however, it appears that there is an increment of value accruing to farm lands when they are made accessible by improved roads.

Land Taxes for Highways.—For a number of years the trend has been away from the practice of levying a road tax upon land to finance the construction of main roads, and the present financial program for highway improvement does not include the use of land taxes for any except the local roads. It has long been the practice, and continues to be so, to depend primarily upon land taxes for the improvement of the strictly local roads such as those commonly designated as *township* and *county* roads. This has been on the theory that although these roads are primarily for land access purposes and consequently the land should bear the principal burden of cost, yet such roads are used to a limited extent for general traffic convenience, and that justifies the application of a portion of the vehicle taxes to such roads.

Vehicle Taxes.—Since the mounting highway expenditures of the period since 1920 have been made necessary largely through the increase in the use of the highways by motor vehicles, it has become the policy to assess various special taxes against road users to secure funds for the improvement of the state and federal highway systems. Since special benefits accrue to traffic from the improvement of the highways, such taxes are logical and fair.

1. *License Fees.*—All states levy a registration fee against motor vehicles. This is in the nature of a readiness-to-serve charge and is exacted upon the same basis from all vehicles that may be expected to use the highways, except those which are owned by some subdivision of the government. In some states the vehicle license fee is a flat tax per vehicle; in others it is levied in accordance with the cost of the vehicle, the horse-

power of the vehicle, or the weight thereof; and in a few cases a combination of these factors is considered in levying the fee. Perhaps the fairest basis upon which to levy this tax would be as a percentage of the cost of the vehicle when new.

2. *Gasoline Taxes.*—The gasoline tax came into the highway financing program when it was found that other taxes failed to produce the revenue necessary for the construction of highways. This tax was first levied on the basis of 1 or 2 cts. per gallon but has been gradually increased in amount until in some states it is now as much as 7 cts. per gallon. This tax, if held to a reasonable rate, commends itself to highway users because the amount thereof will vary with the extent of the use of the highway although not exactly in proportion to such use. In the main this tax has been accepted by the traveling public as fair and equitable.

3. *Commercial Vehicle Taxes.*—The problem of securing from the commercial vehicle a contribution to road funds in proportion to the benefits accruing to such traffic has not yet been solved. The payment of a registration fee and the fuel tax do require such vehicles to contribute to road improvement somewhat in accordance with the extent of use of the highways. Since, however, the commercial vehicle secures more ton-miles of travel per gallon of gasoline than does the automobile, the result of the application of the fuel tax to the commercial vehicle has not produced tax contributions that are equitable between the commercial vehicle and the automobile. Commercial vehicles should pay in proportion to their use on some basis that is fair, but no one as yet seems to know what the fair basis is for such a tax. Perhaps a tax based upon the ton-miles of use of the highways would be the best solution if a suitable administrative system could be devised. Such taxes are already in effect on certain classes of vehicles in some of the states, but the method has not been widely applied.

Theory of Benefits.—The assessment of the cost of street improvements against the adjacent and abutting property is authorized under the general police powers of government. It is recognized that the paving of a street improves the sanitation and drainage and facilitates access to the abutting property. Property near by but not abutting may also be benefited, but such benefits usually are less positive and measurable than those accruing to abutting property.

The benefit, if any, should be evidenced by an increase in the value of the property affected. As a rule such improvements do enhance the value of the property if the expenditures are held to the sum that will provide accommodations for the near-by property. If improvements are of the character required to carry traffic through an area, the benefits will accrue primarily near the termini of the project, and the benefit to the local property will not usually be commensurate with the expenditures. In such cases only a part of the cost should be assessed to the adjacent property.

The logical basis for levying such special assessments, perhaps, would be as a percentage of the increase in value due to the improvement. The difficulty lies in determining at the time the assessment is made what the effect of the improvement will be on the value of the property. Value at any time is based on the estimated present worth of the probable future services to be rendered by the improvement. This future service may be a cash return in the form of a rental, or it may be in the satisfaction enjoyed by the owner from his use of the premises. It is almost axiomatic that street improvements increase the value of the adjacent property, but tangible evidence of this increase can be secured only by receiving higher rentals or deriving increased satisfaction from the use of the property. Neither of these effects will ordinarily be noted immediately after a street is paved, although eventually rental rates and sales prices do reflect any benefit. At the time of levying the special assessment there will generally be no tangible evidence of change in value, and to levy the assessments on the basis of the increase in value due to an improvement is quite impossible—a practical matter that has been discovered where it has been tried.

The magnitude of the benefits due to street improvements is greatest at the street line and diminishes gradually as the distance from the improved street increases. There have been attempts to formulate a law of diminishing benefits applicable to this condition, but the circumstances surrounding each project will influence the effect of the improvement on values and benefits; therefore no law or rule seems to be universally applicable. It is believed that the benefits decrease as some power greater than unity of the distance from the street to the parcel of property in question. For most projects of a simple character it will not be far out of line to assume that the benefits diminish as the

square of the distance, but in some zone assessments it has been assumed that the decrease was as the cube of the distance. For each project the assessor must fix the rate of diminishing benefits in accordance with his estimate of what is fair in that particular situation.

The principle of benefits is readily adapted to ordinary assessment projects where the cost of an improvement is assessed on the area between the street that has been improved and a line midway to the next parallel (or approximately parallel) street or otherwise limited to a comparatively narrow strip on either side of the improvement. But when extensive rebuilding, boulevarding, or relocating is undertaken it is probably equitable to spread the cost over a considerable area adjacent to the improvement. Here there are variations in benefit not only along a line transverse to the improved street but also from block to block along the improved street itself. Quite commonly these projects (often called "superhighways," or "express highways") are built to provide for travel from outlying areas in a city to the central business district, and the benefit is primarily to the traffic. The adjacent property lying intermediate between the residential and the business district is little benefited and may even be damaged by the improvement. In all projects of this type such special assessments as are levied ought to be spread on the basis of the true benefits throughout the assessment area so far as fallible human judgment will permit.

In theory, then, the special assessment is to be spread on the basis of the special benefits conferred by the street improvement. But the size and shape of the parcels of land, the nature of the business conducted thereon, the area of the block, and many minor physical features of the street and alley layout are involved in the determination of special benefits, and the equitable assessment of the cost of street improvements is not susceptible of reduction to formula. Nevertheless, various formal methods are employed for spreading assessments, and these serve admirably to distribute equitably the rate of benefit after the basis of distribution has been determined upon. In every instance the final assessment is arrived at by a judicious review and modification of the results of a mathematical distribution of benefits.

The distribution of benefits is computed in various ways, depending partly upon the local laws relating thereto and partly

upon the judgment of the assessing officer as to what constitutes a suitable basis for the particular project. In every case the purpose is to assess the costs in accordance with the benefits and with equity to the various properties involved.

Items of Cost Included in Special Assessments.—Most of the state laws relating to special assessments permit the addition of the cost of all incidental work to the cost of the pavement and the total thus obtained, or some predetermined part thereof, to be assessed against the property within the statutory distance of the improvement. The cost of storm inlet changes, manhole changes, incidental work, legal expenses, and sometimes engineering and inspection are considered to be essential parts of the cost of the improvement.

In computing the total amount to be assessed for the pavement on a street, it is customary to compute the cost of each section of substantially identical construction, whether it is a part of a block or many blocks. If the pavement is of the same design and dimensions throughout, differences in the number of items of special or incidental work in the various blocks need not be considered to constitute a change in type. Whether or not the cost of the part of the work at street intersections is to be included depends upon local laws or customs. A part of the total cost may be paid from general or special funds, and the remainder specially assessed, or the total may be assessed.

In the following sections there will be presented a number of methods that have been employed for spreading assessments. These are mathematical in character and make no provision for modifying the assessment for parcels of land that are obviously benefited either more or less than the average. Adjustments for these cases must be made by judgment. The assessor must always recognize the need for a method that will on any street assess exactly the same sum against lots of the same size and similarly located with reference to the improvement and likewise assess either more or less against other lots of different dimensions or differently located. This requires a mathematical process for the initial distribution of the costs.

Front Foot Rule for Assessments.—In many cities, only the abutting property is assessed, and the cost is spread in proportion to the length of frontage of each lot or parcel of land on the paved street. This rule is, of course, simple of application but does not take into account possible variation in the depth of the

TABLE I.—ILLUSTRATING THE ZONE-AND-AREA METHOD OF ASSESSMENT

Assessments for B Avenue

Block	Lot	Parcel	Area, sq. ft.	Rate	Assessment	Remarks
			Zone 1			
1	11	f	2,500	0.179293	$ 448.23	B Avenue Rates: Zone 1—65 per cent. Zone 2—25 per cent. Zone 3—10 per cent.
..	..	..	.....	....	....	
1	10	a	2,500	....	448.23	
..	..	..	.....	....	....	
1	7	a	3,750	....	672.34	
1	6	..	5,000	....	896.47	
..	..	..	.....	....	....	
2	7	a	2,500	....	448.23	
..	..	..	.....	....	....	
2	3	a	5,000	....	896.47	
..	..	..	.....	....	....	
			Zone 2			
1	11	a	2,500	0.068960	172.40	
..	..	..	.....	....	....	
1	10	b	2,500	....	172.40	
..	..	..	.....	....	....	
1	7	b	3,750	0.068960	258.60	
1	5	..	5,000	....	344.80	
2	9	a	2,500	....	172.40	
..	..	..	.....	....	....	
2	3	b	5,000	....	344.80	
..	..	..	.....	....	....	
			Zone 3			
1	12	f	2,500	0.027584	68.96	
..	..	..	.....	....	....	
1	10	d	1,250	....	34.47	
1	7	c	3,750	....	103.43	
1	4	..	5,000	....	137.91	
..	..	..	.....	....	....	
2	3	c	5,000	....	137.92	
..	..	..	.....	....	....	

Assessment for A Street

Block	Lot	Parcel	Area, sq. ft.	Rate	Assessment	Remarks
			Zone 1			
1	13	a	5,000	0.145003	$ 725.02	A Street Rate: Zone 1—50 per cent. Zone 2—25 per cent. Zone 3—15 per cent. Zone 4—10 per cent.
1	12	a	2,500	0.145003	362.51	
..	..	..	.....	....	....	
2	9	a	2,500	....	362.51	
..	..	..	.....	....	....	
2	11	a	7,500	0.145003	1,087.52	
			Zone 2			
1	13	b	5,000	0.072501	362.51	
..	..	..	.....	....	....	
2	9	b	2,500	....	181.25	
..	..	..	.....	....	....	
2	11	b	7,500	0.072501	543.76	

TABLE I.—ILLUSTRATING THE ZONE-AND-AREA METHOD OF ASSESSMENT —Continued

Assessment for A Street

Block	Lot	Parcel	Area, sq. ft.	Rate	Assessment	Remarks
			Zone 3			
1	13	c	5,000	0.043500	217.51	
1	12	d	2,500	....	108.75	
2	7	c	2,500	....	108.75	
2	6	a	7,500	0.043500	326.25	
			Zone 4			
1	9	a	7,500	0.02900	217.51	
1	10	b	2,500	....	72.50	
2	6	b	7,500	0.02900	217.51	

several lots, which would be desirable if the lots were not of about equal depth.

Area Method of Assessment.—Another simple method is to spread the assessment according to the area of each lot or parcel of land. This takes account of the differences in size of the lots but does not compensate for differences in frontage on the improved street, which would be desirable if the lots were not of about the same width throughout the area assessed.

Zone-and-area Method.—In this method the area to be assessed is divided into equal zones, each extending the length of the pavement for which the assessment is being prepared. A rate of assessment is then determined for each zone. In the example given herein (Fig. 5) the area to be assessed for the improvement of A street is divided into four zones, and the rates for the several zones are 50, 25, 15, and 10 per cent, respectively. This means that the zone nearest the street bears 50 per cent of the total cost of the improvement, which is distributed to the various parcels of land in the zone in proportion to their respective areas. The area to be assessed for the improvement of B Avenue is divided into three zones, bearing respectively 65, 25, and 10 per cent of total cost. Table I shows in outline the method of spreading the assessment.

This method is applicable to blocks that are divided in a regular manner into rectangular areas of substantially equal

depth measured perpendicularly to the street or into areas that are even multiples of some common unit depth.

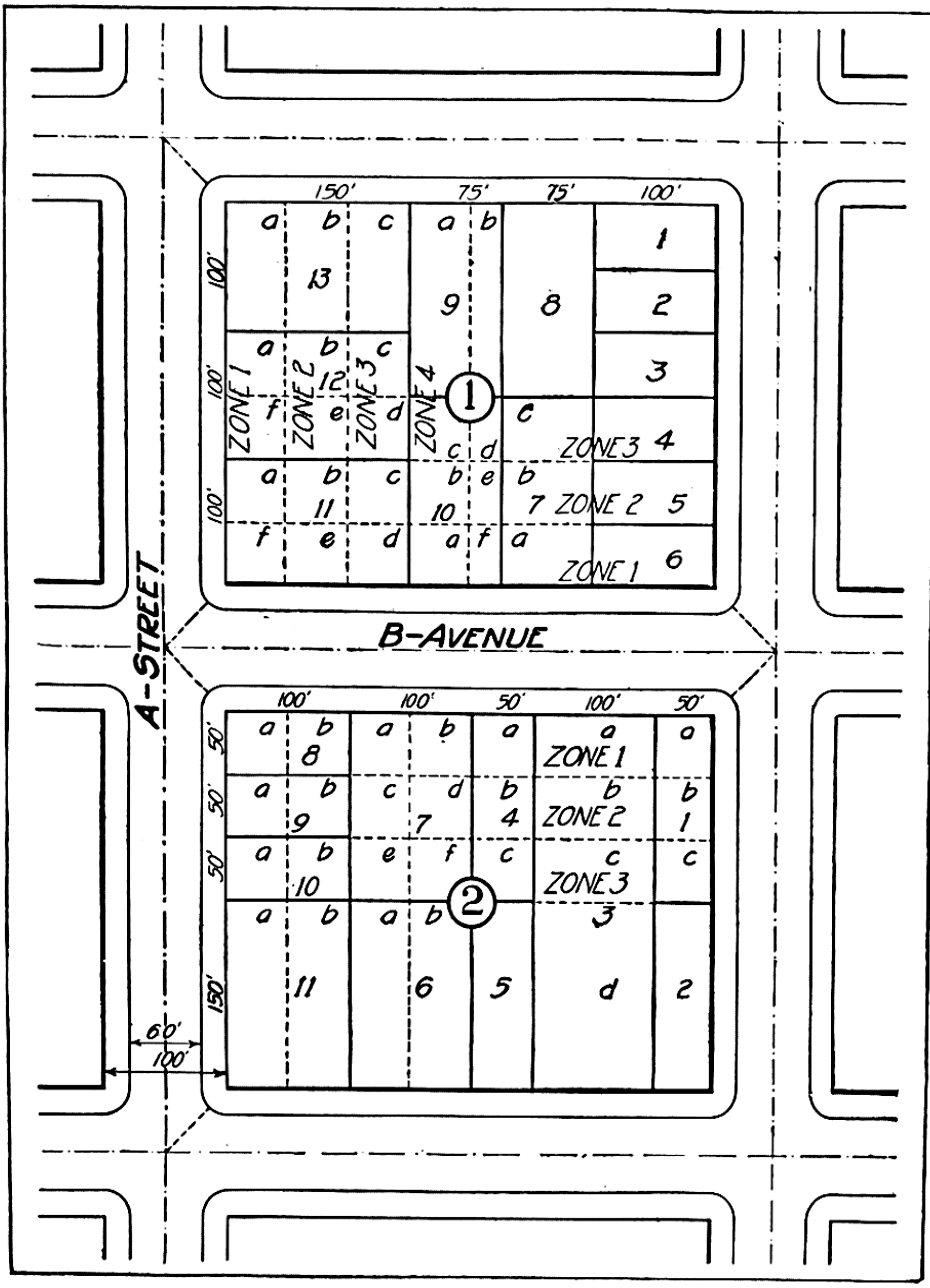

FIG. 5.—Plat for zone-and-area method of assessment.

The area to be assessed may be divided into any desired number of zones, and the respective rates on the zones fixed in accordance with the judgment of the responsible official body.

Benefit Factor Method.—The zone-and-area method of spreading assessments is based on the theory, which is also accepted in

real estate valuation, that benefits from street improvements decrease as some power greater than unity of the distance from the street. Some authorities consider the benefits to decrease as the square of the distance, and some as the cube of the distance. In the zone method the benefits are assumed to be constant in each zone, which is obviously not true, but the lot layout is often of such a character that no injustice results from the assumption. In other instances the lots and parcels of land are laid out in such a way that more equitable results are obtained by taking

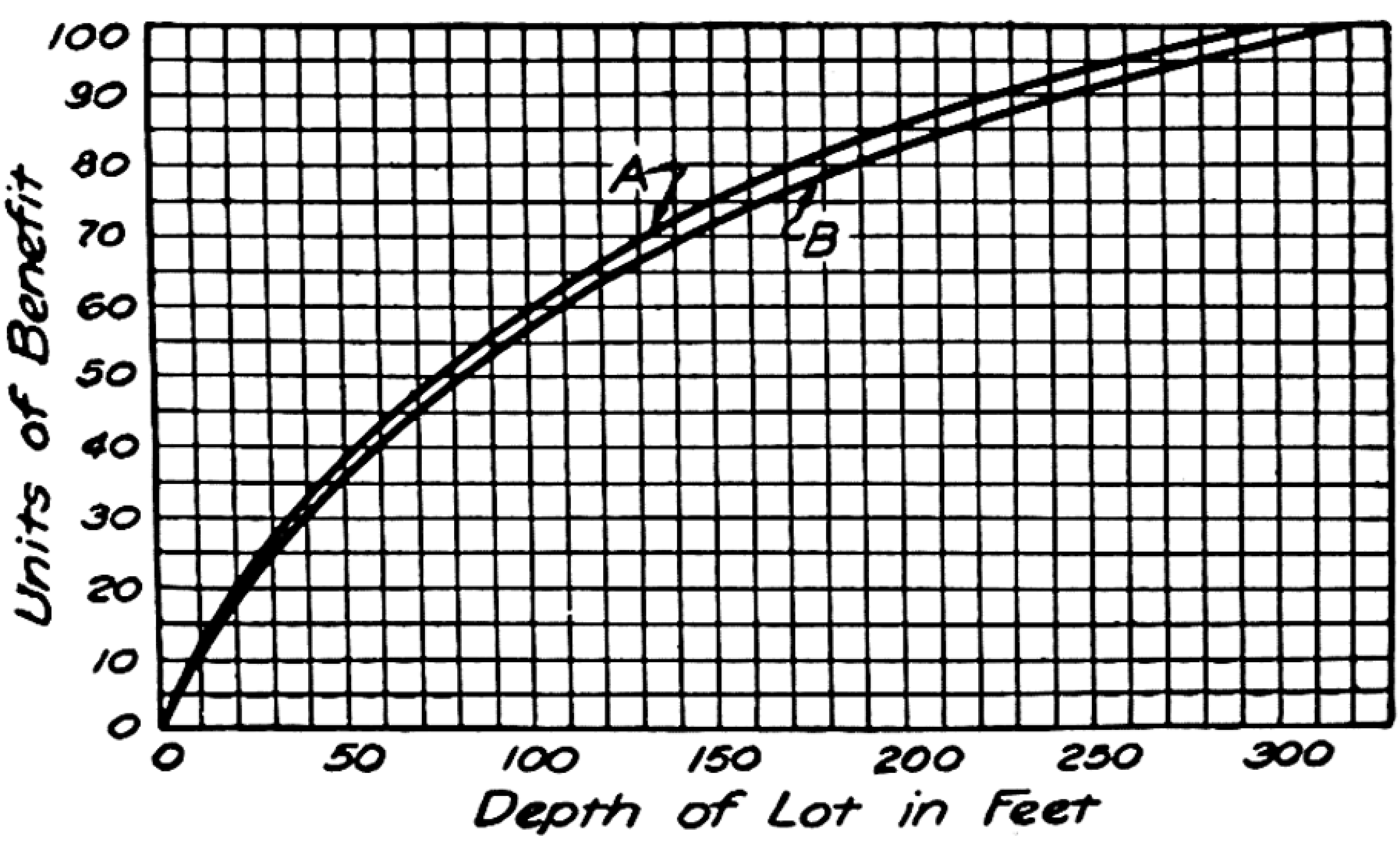

FIG. 6.

account of the rate of diminishing benefits as the distance from the improvement increases. This is accomplished by determining the benefit for each parcel of land from a benefit curve drawn to represent the assessor's opinion of the rate of diminishing benefits for the improvement for which he spreads the assessment.

There is shown in Fig. 6, Curve *A*, a benefit curve for an assessment area 300 ft. wide (block 600 ft. long), platted on the assumption that the benefits diminish 50 per cent in the first one-fourth of the distance, 25 per cent in the second one-fourth of the distance, and 15 per cent in the third one-fourth of the distance. Curves showing other rates of diminishing benefits and for other widths of assessment area may be prepared to comply with existing conditions, and the curve shown herein is illustrative only, although it has been used for many special assessments. Figure 7 is the plat for an assessment district, and

TABLE II.—ASSESSMENT FOR HODGE AVENUE

Lot	Block	Rate	Length	Unit of benefit (3) × (4)	Unit assessment	Assessment (5) × (6)
1	2	3	4	5	6	7
6	13	72.5	57.0	4,132.5	$0.0921436	$ 380.78
..	..	....	......	....	....	....
1	14	72.5	57.0	4,132.5	....	380.78
..	..	....	......	....	....	....
1	15	12.5	213.65	2,670.62	....	246.08
..	..	....	......	....	....	....
1	18	11.2	147.5	1,652.0	....	152.22
..	..	....	......	....	....	....
1	19	72.5	59.0	4,277.5	....	394.14
..	..	....	......	....	....	....
3	20	72.5	68.0	4,930.0	....	454.27
..	..	....	......	....	....	....
6	21	72.5	60.0	4,350.0	....	400.83
..	..	....	......	....	....	....
1	22	43.5	142.0	6,177.0	....	569.17
..	..	....	......	....	....	....
Totals....	..	....	......	161,366.65	....	$14,866.14

Lot	Block	Rate	Length	Unit of benefit (3) × (4)	Unit assessment	Assessment (5) × (6)
1	2	3	4	5	6	7
3	15	1.2	86.6	103.92	$0.0870784	$ 9.05
..	..	....	......	....	....	....
12	15	10.6	213.65	2,264.69	....	197.21
..	..	....	......	....	....	....
4	18	19.2	147.5	2,832.00	....	246.61
..	..	....	......	....	....	....
7	22	28.6	142.0	4,061.20	....	353.64
..	..	....	......	....	....	....
Totals....	..	....	......	86,118.29	....	$7,760.28

Table II shows the method of applying the curve to the assessment of the cost of the pavement on Hodge Avenue. A somewhat different procedure must be followed in spreading the assessment for the work on the diagonal street, Northwestern Avenue. The most exact method would be to divide each lot

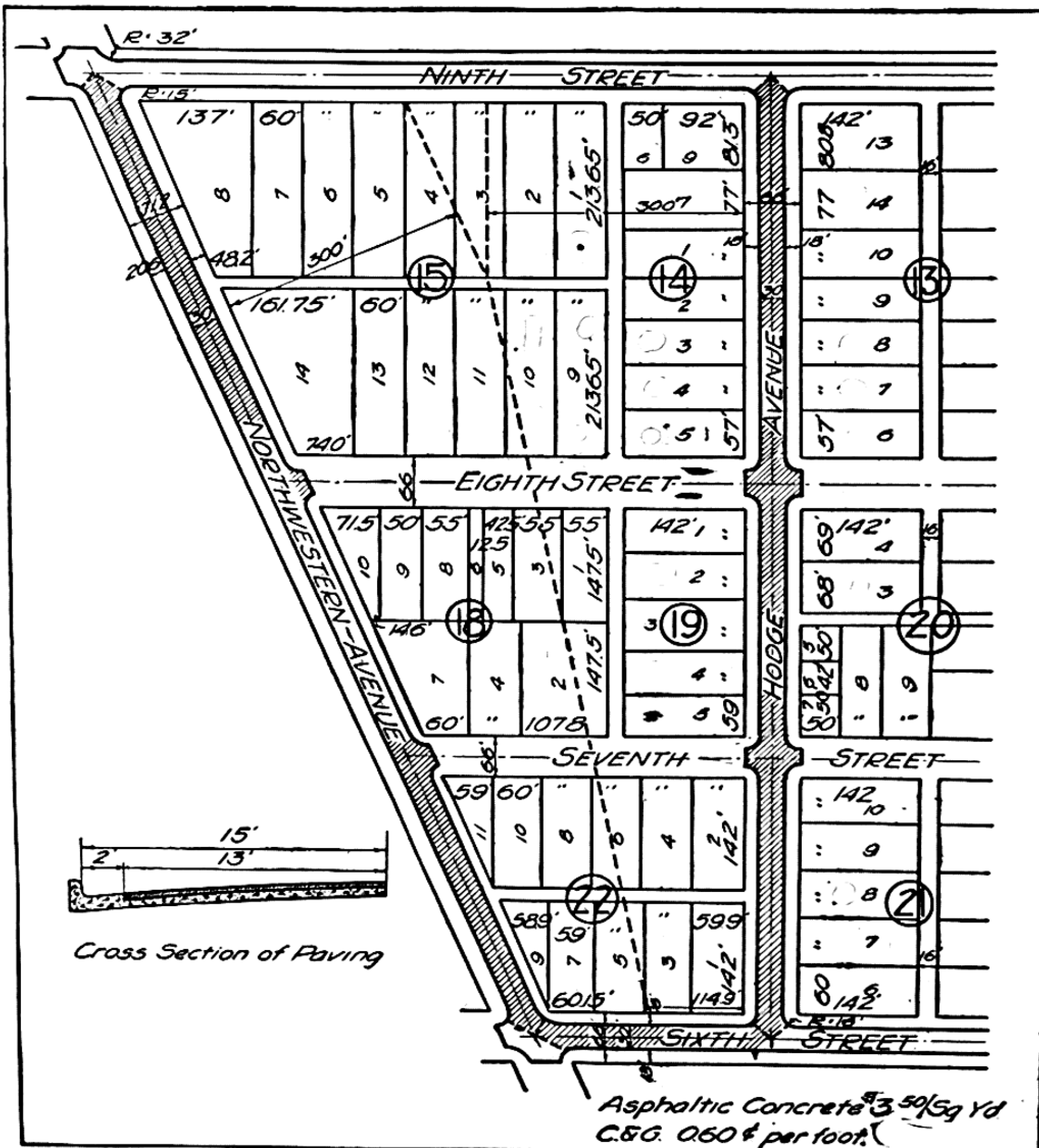

FIG. 7.—Plat for assessment by benefit factor method.

into strips 1 ft. wide parallel to the street, but this involves an endless amount of labor in that particular instance. Another method which is sufficiently accurate for most projects is to draw a new benefit curve, on the same basis as the old, but for a width of zone determined by a perpendicular to the lot bound-

ary lines in the zone. Such a curve is shown in Fig. 6, Curve *B*. The assessment may now be spread as before. The triangular-shaped lots may be assumed to have a depth equal to the average width or, if very precise results are desired, may be divided into strips 1 ft. wide parallel to the regular lots in the zone. The method of assessment for Northwestern Avenue is also shown in Table II.

Assessing Irregular Shaped Parcels.[1]—The method employed in the foregoing examples is accurate for rectangular-shaped parcels of land and nearly so for parcels that are nearly rectangular but will introduce rather sizable inequalities if the parcels are triangular or are sectors. When the zone to be assessed contains a few irregular shaped parcels, each may be divided into thin slices parallel to the pavement, and the assessment determined by summing up the assessments of the slices, which are assumed to be rectangular. If there are many irregular parcels in a project, time will be saved in working out the assessment by using a mathematical method of determining the intensity of benefit for each irregular parcel of land. The method is also exact for rectangular-shaped parcels, but the benefits to these can more quickly be determined from curves or tables.

The first step in the mathematical process is to adopt an equation that represents the desired rate of diminishing benefits. A few trials will suffice to develop an equation that approximates closely any arbitrarily assumed rate. For example, Curve *A*, Fig. 6, is represented very closely by the equation

$$r = \frac{600y - y^2}{900}, \tag{1}$$

where r is the intensity of the benefit at any distance from the improvement, and y is the distance from the property line of the improved street to the point at which the intensity of the benefit is being determined.

In order to illustrate the application of Equation (1) to computations of assessments, two cases are considered.

Case I.—Base of a triangular parcel or the arc of a sector, toward the street for which the assessment is being made, as illustrated in Fig. 8.

[1] Contributed by Ralph A. Moyer, Associate Professor of civil engineering, Iowa State College.

Solution for Case I.

$$\text{Let } r = \frac{600y - y^2}{900} \text{ (from Equation (1)),}$$

where r is in units of benefit, and y in feet. The benefit on the elementary area ldy is ldr. The total benefit for the entire parcel is Σldr.

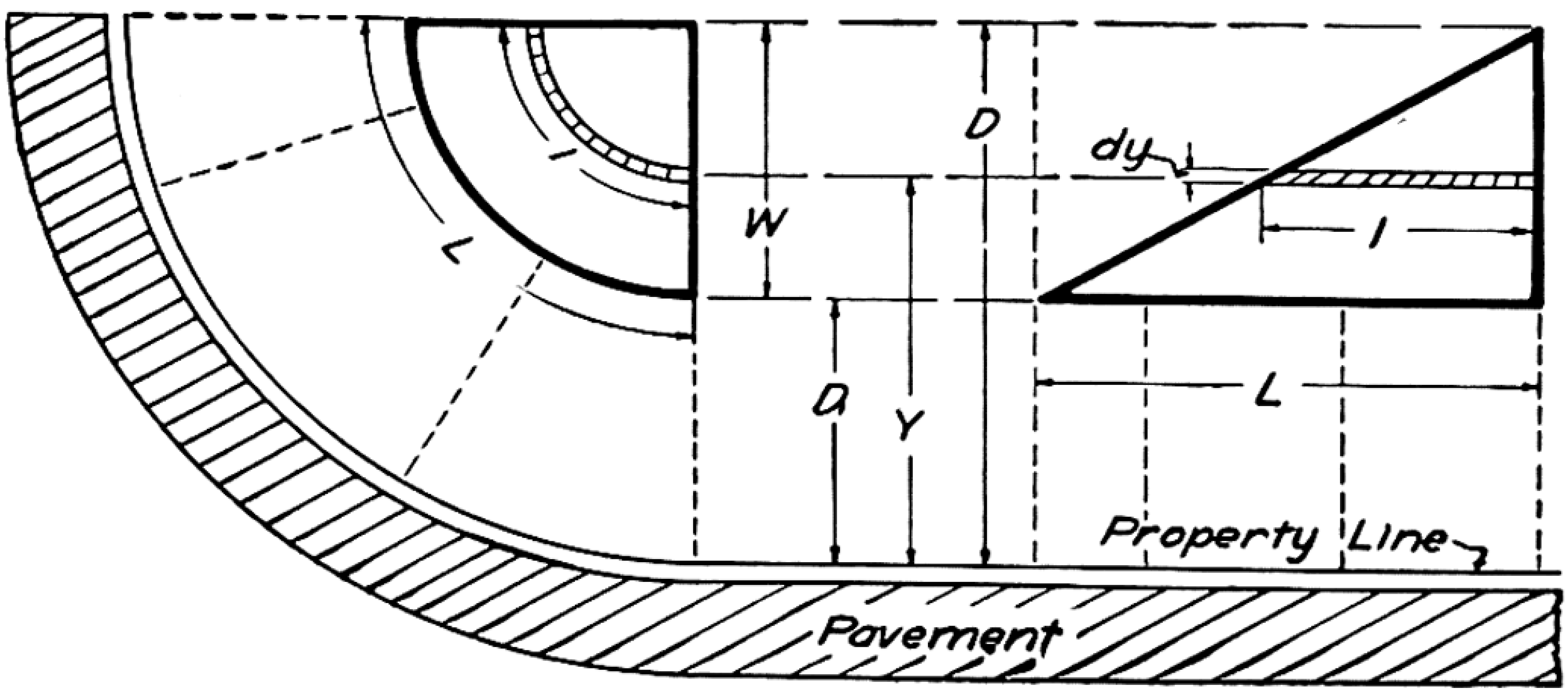

Fig. 8.

Then

$$dr = \frac{1}{450}(300dy - ydy). \tag{2}$$

From the geometry of Fig. 8 it is readily seen that

$$l = \frac{L}{W}(D - Y); \tag{3}$$

and by assembling the factors as defined above,

Total benefit for either parcel in Fig. 8

$$= \Sigma ldr = \frac{L}{450W}\int_{D_1}^{D} 300Ddy - (300 + D)ydy + y^2dy$$

$$= \frac{L}{2{,}700}[900W - 3WD_1 - W^2]. \tag{4}$$

Case II.—Apex of a parcel toward the street for which the assessment is being made, as shown in Fig. 9.

Solution for Case II.—By a method analogous to that employed for Case I, it is found that

Total benefit for either parcel in Fig. 9

$$= \Sigma ldr = \frac{L}{450W}\int_{D_1}^{D}(300dy - ydy)(y - D_1)$$

$$= \frac{L}{2{,}700}(900W - 3WD_1 - 2W^2). \qquad (5)$$

Since the quantities W, D_1, and L are known for each parcel, the units of benefit can be calculated by substitution in either

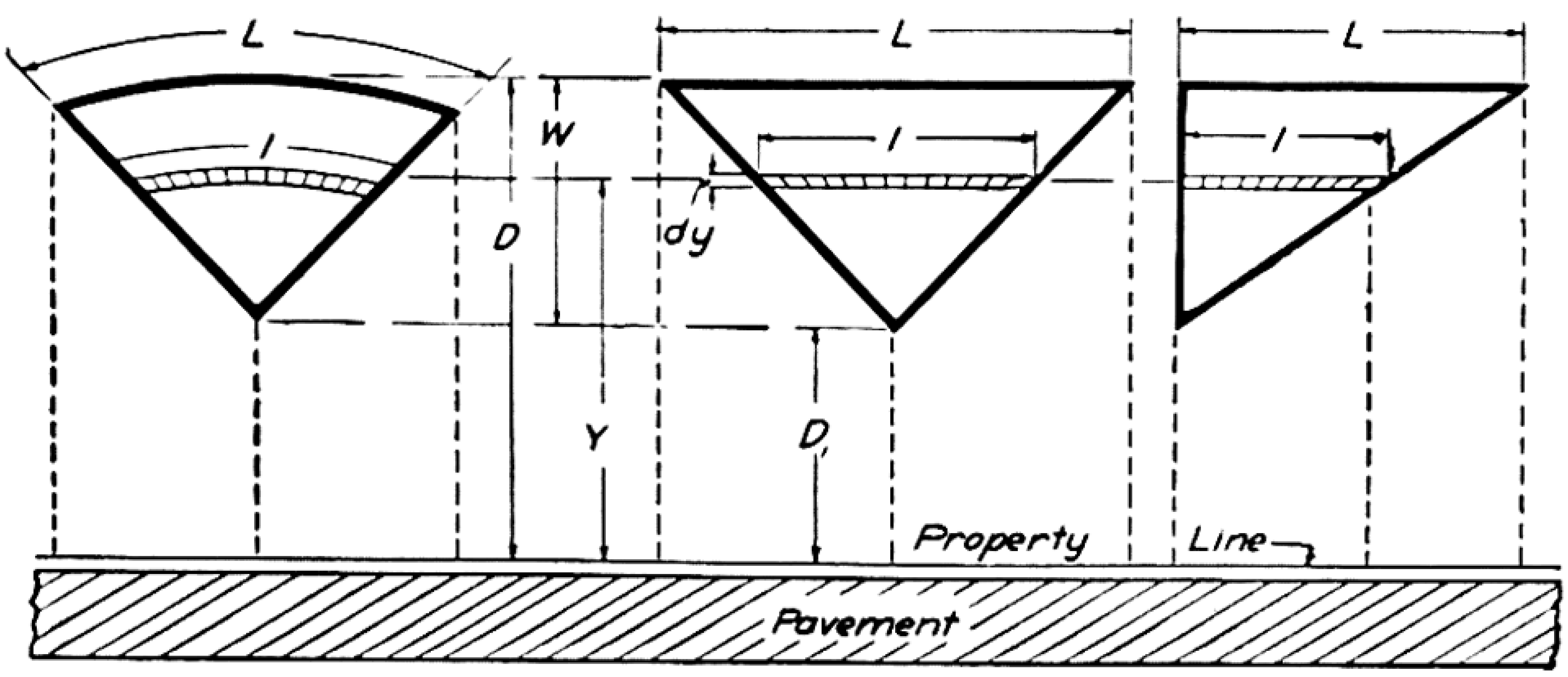

FIG. 9.

Equation (4) or Equation (5), according to the position of the parcel.

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CHAPTER II

SURVEYS AND PLANS FOR ROADS AND PAVEMENTS

BY LOWELL O. STEWART[1]

I. ROAD SURVEYS

The completeness and detail with which a road survey should be made depends upon the character of the improvement contemplated and whether the work is to be done by contract or by force account. Surveys by various engineers differ in many respects even though made for the same class of construction, and it is not possible to standardize them because of that fact. So far as the actual work of making the surveys is concerned, no principles are involved that are not involved in making surveys for other engineering projects. As with other surveys, the information obtained and the method of tabulation are adapted to the needs of the work in hand.

For the purposes of discussion it is convenient to divide all surveys into four general classes: reconnaissance surveys; surveys for earth roads to be graded by force account; surveys for hard-surfaced rural highways; and surveys for paving. These represent four broad classes of surveys which inevitably overlap to some extent.

Reconnaissance Surveys.—These surveys are of two kinds, the first of which is identical in purpose with the reconnaissance survey made in railroad work. Its purpose is to find the best location or route for a new highway, or relocation of an old one. This type of survey needs no explanation.

The second type of reconnaissance survey is one that has been adapted primarily to highway work and has for its purpose the selection, from among several roads, of the most feasible for improvement. It involves the examination of the road to secure data for the preparation of a map showing the various physical characteristics such as bridges, culverts, embankments, cuts, hills, the apparent width of right-of-way, the character of the

[1] Professor and head, department of civil engineering, Iowa State College.

soil, and any other fact that would have a bearing on the selection of a route.

Apparently such a survey differs widely from the ordinary reconnaissance and yet it serves substantially the same purpose, namely, the selection of a tentative route. The survey is made by walking or driving over the road. Distances are obtained by the pedometer or odometer and checked by intersecting roads and survey monuments when these are encountered. The notes are kept in the ordinary field book upon which the route has already been platted from existing maps so that it remains only to fill in the detailed information.

Surveys for Earth-road Construction.—For earth-road grading, especially blade-grader or elevating-grader work to be done by force account, it is not always necessary to make complete detailed surveys. Grading done by force account is not usually finished very closely to grade, nor will it be necessary to balance earthwork on the plans, because adjustments can be made in the field as the work progresses. The object of this kind of a survey is to give an approximate grade for the outfits to work to so that drainage will be assured and reasonable gradients result. The transit survey usually consists in running a center line and locating fence lines, bridges, culverts, and other physical characteristics relative to this line and to the adopted stationing. Houses, entrances to farms, trees, pole lines, bridges, and similar physical characteristics are indicated, as well as the type of soil encountered.

The level survey consists in running a center-line profile, and possibly a profile in each ditch. If a section is encountered where it appears that a considerable change in grade will be desirable, cross-sections are taken at 100-ft. stations and at intermediate breaks in grade, if such occur.

This kind of a survey is suitable for force-account grading and is permissible where funds are not to be had for a more complete detailed survey. It is used especially on earth-road grading or low-grade surfacing where the local conditions are such that the requirements are not exacting.

Complete Detailed Surveys.—Complete detailed surveys are a prerequisite to the preparation of plans for contract highway work except that which is performed on the cost-plus basis. These surveys are made in the following successive steps, which have become recognized as the usual practice in highway work:

1. The preliminary survey, which includes: special topographic surveys of relocations; the center-line survey; topographic surveys for bridges and grade separations at intersecting highways and railroads.
2. The right-of-way survey.
3. The construction survey.
4. The final-estimate survey.

THE PRELIMINARY SURVEY

The modern methods of highway transportation have necessitated the relocation of portions of many of the older roads before they are modernized. In a few instances, a new location may be selected on the basis of a study of an existing topographic survey of the area to be traversed, for which purpose maps of the United States Geological Survey are suitable. But since so small a portion of the United States has been mapped, a topographic survey will in most instances be necessary in planning relocations. As a general rule, either the plane-table or the transit-stadia method will be used for this survey, the scale adopted depending upon the size and configuration of the area to be mapped.

Where the final location is to be selected from a number of feasible locations, the engineer likes to make sure that all of the possibilities have been investigated. It is better to survey several lines that are later eliminated from consideration than to overlook one that later proves to be the most satisfactory.

All data obtained in these surveys should be clearly identified in the notebooks, and the surveyed lines should be shown on a sketch map of the entire project, drawn approximately to scale. The notes for each line should be indicated by means of a key number or letter, such as "line *A*," "line *B*," "line 27*C*," which should appear both on the map and in the notes.

THE CENTER-LINE SURVEY

The system of stationing and the datum for elevations should be established before the survey is begun. The stationing should be carried continuously from the beginning to the end.

Stationing.—When alternate lines are surveyed for a part of the route to provide data for possible relocations, the stationing on the alternate line will be referenced to the primary line by means of station equations. Assume that the primary line

is called A and an alternate line B, and the alternate line begins at Station 35 + 17.6 and ends at Station 65 + 38.8 of line A, and the alternate line is 2274.6 ft. in length. The appropriate station equations are:

$$B\text{-line Sta. } 0 + 00 = A\text{-line Sta. } 35 + 17.6.$$
$$B\text{-line Sta. } 22 + 74.62 = A\text{-line Sta. } 65 + 38.8.$$

The equality sign (=) is used to mean "coincides with."

The Survey.—The several operations which form the complete center-line survey may be listed as follows:

1. Staking the center line and taking the topography.
2. Establishing the bench marks.
3. Taking the cross-sections.

Staking the Center Line.—Each 100-ft. station on the center line is marked by a stake (called a "hub") set on line. If the survey line is on the traveled part of a road, these hubs may be large spikes thrust through a piece of red cloth and driven down until the head is about 3 in. below the road surface so as to escape the road maintainer. Where the line lies in fields and untraveled sections of road, a wooden stake is preferred. A marker stake, carrying the station number, should be driven near each hub, or at an indicated rectangular offset when the hub is on a traveled road.

In some instances, it may be known in advance that the construction line will follow exactly the preliminary line. Where that is true, it is well to place offset hubs with tacks on one or both sides of the road, at the time the preliminary line is run. These would be placed at an offset of 25 or 30 ft., opposite each 100-ft. station. Alongside each offset hub there is driven a marker stake upon which the station number is marked.

Unless other considerations make it impracticable, it will be desirable to run the surveys from west to east or from north to south, so that the plans may be oriented to conform with standard practice, that is, with north at the left or top.

Referencing Transit Points.—Each point on the line at which the transit is set up should be referenced very carefully. Many engineers prefer the method in which four reference points are selected in such a manner that the transit point is at the intersection of the X formed by connecting diagonally opposite reference points. These reference points may be permanent objects such as trees, boulders, wall corners, and the like, with

hubs set near by, or the hubs may be used alone, if no permanent object is convenient. Both types of reference point are desirable, the former for their permanence, the latter for the ease with which a transit may be set up over them and pointed to the elusive transit point.

The notes should include a sketch of the layout at each transit point showing the distances and direction from the transit point to the reference points and the kind of reference points.

Each transit point must be readily visible from the preceding and following transit points, but a half mile is about the maximum satisfactory distance between transit points, even though the instrument man may be able to see farther from some of the points.

Intersection Angles.—The intersection angle at each point of intersection (P.I.) of diverging sections of the line is measured and recorded. It may be measured by deflection or by measuring clockwise from the backsight, but one method or the other should be followed throughout the survey. Doubling the angle will guard against errors. Compass bearings are a valuable aid in detecting mistakes of a half degree or larger.

Horizontal Curves.—At points of intersection where the angle of intersection is larger than two degrees, a horizontal curve should be staked out with the stationing carried continuously around the curve. The type of curve and the degree will be fixed by the conditions and the standard practice of the organization for which the survey is being made.

Topographic Features.—As the party proceeds along the center line it should locate all objects that are likely to be of importance in connection with the design of the proposed improvement. This will generally include everything within a strip 150 ft. wide on each side of the center line, and certain additional data regarding drainage areas and the like. With a little practice, a man can pace the distances on the center line (between 100-ft. hubs), while distances to the right or left of the center line will be taped by others of the party who will also measure buildings and other structures. The following list gives the most of the objects that will be recorded in the ordinary highway survey:

1. Corner stones.
2. Section and quarter-section lines.
3. Division fences.
4. Side roads and the angle at which they intersect the main road.
5. Buildings.

6. Rows of trees, with notes as to kind, usefulness, and size.
7. Existing tile lines, with notes as to conditions, size, and outlet.
8. Field entrances and driveway entrances.
9. Sidewalks.
10. Culverts and bridges, with station of each end of large bridges.
11. Telephone lines and transmission lines.
12. Guard fence.
13. Retaining walls.
14. Gravel pits and quarries.
15. Limits of each property ownership and name of owner.

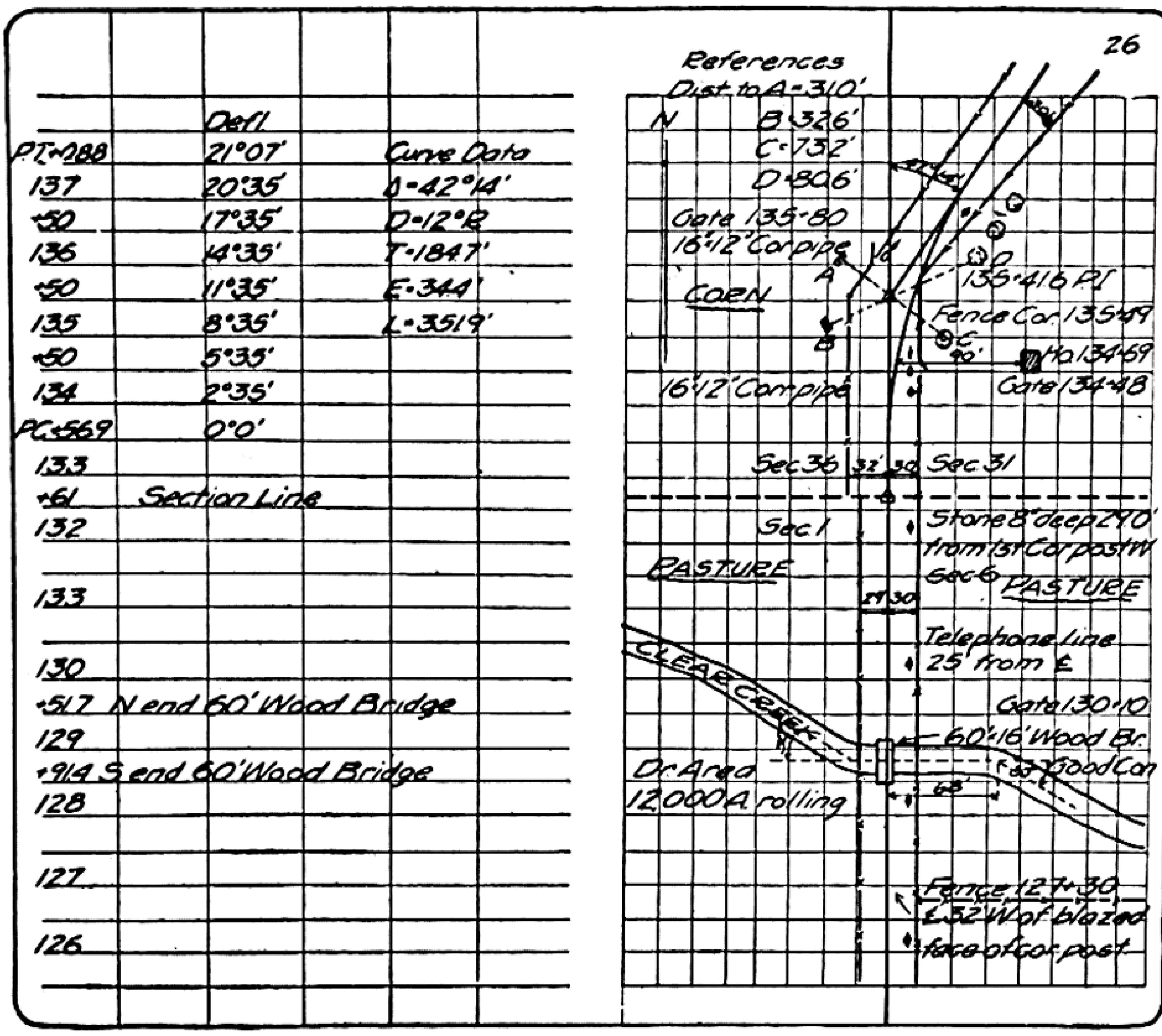

FIG. 10.—Typical transit survey field notes.

NOTES.—A standard form of notes is shown in Fig. 10. The sketch is placed on the right-hand page and station numbers with notations are on the left. Both the sketch and notes begin at the bottom of the page, an arrangement which enables the notekeeper to record his data and draw his sketches with a minimum amount of mental juggling. The notes and sketches should never be crowded.

Corner Stones.—Survey monuments have been established for many purposes by various governmental and local agencies. Probably the earliest were those of the government land surveyors who measured and marked the land upon the ground. In the eastern states, these surveys were made by metes and bounds.

In the states farther west, the rectangular system of the General Land Office was used. As the land was settled and sold, resurveys and new surveys were requested and carried out. These surveys disclosed some errors in the original work. This impelled the General Land Office to establish the general rule that all corners must stand as actually set by the original surveyor, even though the position does not agree with the notes. The work of later surveyors has been to retrace the lines and reestablish and remark the original corners as nearly as possible.

The records of these original surveys were kept in the office of a surveyor-general who had charge of one or more states. When the surveys of a state had been completed, those records were turned over to the state officials, who held them open to reference by anyone. Local governmental units, such as the counties, had need for many of these data, so copies of the descriptions were furnished to each county. In most states, the county records now afford the only up-to-date descriptions of monuments, because of the fact that all surveys subsequent to the original government surveys are recorded in the county recorder's office.

The engineer who is engaged in making highway surveys has frequent occasion to locate the original survey monuments along a highway. These have, in many cases, been exposed by the wear of traffic or by grading operations, and in other cases, are buried and must be uncovered. Normally these monuments should be found at half-mile or lesser intervals, but the exact location of intermediate monuments is not readily determined in all cases because the original lines were not run with transit accuracy.

If a preliminary examination fails to unearth the desired monument, a systematic search must be inaugurated. The county records should be examined to ascertain just where the monument should be found. Additional information may often be secured from local engineers or surveyors, road maintenance crews, or residents of the neighborhood.

Datum and Bench Marks.—Sea-level datum is used if it has been established near the line to be surveyed. When neither sea-level nor local datum is available, an assumed datum is selected which is high enough to insure that no negative elevations will develop on the survey. A series of bench marks is established on the basis of the adopted datum. Bench marks should be in places that are not likely to be disturbed before or

during construction operations. Objects already in place are suitable and convenient, as a rule. Bridge abutments and wing walls, head walls of concrete culverts, concrete steps, and large boulders are good. A railroad spike driven into the root of a tree is satisfactory. In open country, pieces of iron pipe about 4 ft. long may be driven near the fence line.

In general, bench marks should be placed at quarter-mile intervals, and closer in rough country. One should be placed near each bridge, culvert, or similar structure. Each bench mark should have a number, beginning with *N*-1 at the zero end of the line, and each should be described very carefully to facilitate locating it when it may later be needed.

All bench-mark elevations should be checked very carefully. On short projects it will be quite satisfactory to check bench-mark elevations when the cross-sections are taken. On lines 1 mile or more in length, a separate set of check levels is desirable to obviate bothersome corrections in the cross-section elevations. In either case, the elevations of bench marks should conform to standard practice, which is that the difference in successive determinations of the elevation of two bench marks should not exceed $0.05\sqrt{D}$, where D is the distance in miles between bench marks.

Cross-sections.—Elevations on lines transverse to the center line, called cross-sections, should be taken at each 100-ft. station and at intermediate places where there are pronounced changes in the topography, whether the change is on the center line or at the side. The cross-sections should be extended to the right-of-way lines and as much farther as conditions warrant. This outer limit at any particular station will depend upon the depth of cut or fill and the width required by the cross-section of the proposed improvement. The engineer should visualize each situation with its probable grade line to make sure that he has extended his cross-sections to the proper limits.

Additional cross-sections should be taken at the ends of bridges and wing walls and at culverts. If the culvert is permanent, the distance back-to-back and the elevations of head walls should be recorded, to give data for determining the allowable height of fill. Railroad and trolley crossings will require special cross-sections or special surveys.

Intersecting Roads and Driveways.—Center-line profiles and cross-sections should be taken on side roads and driveways so

that the designer may establish a suitable approach. In level country, a profile extended about 300 ft. will suffice. But where the side road must be graded to fit the main road, enough cross-sections should be taken to permit the balancing of cuts and fills on the side road. Usually the center-line profile is the only datum needed for a driveway.

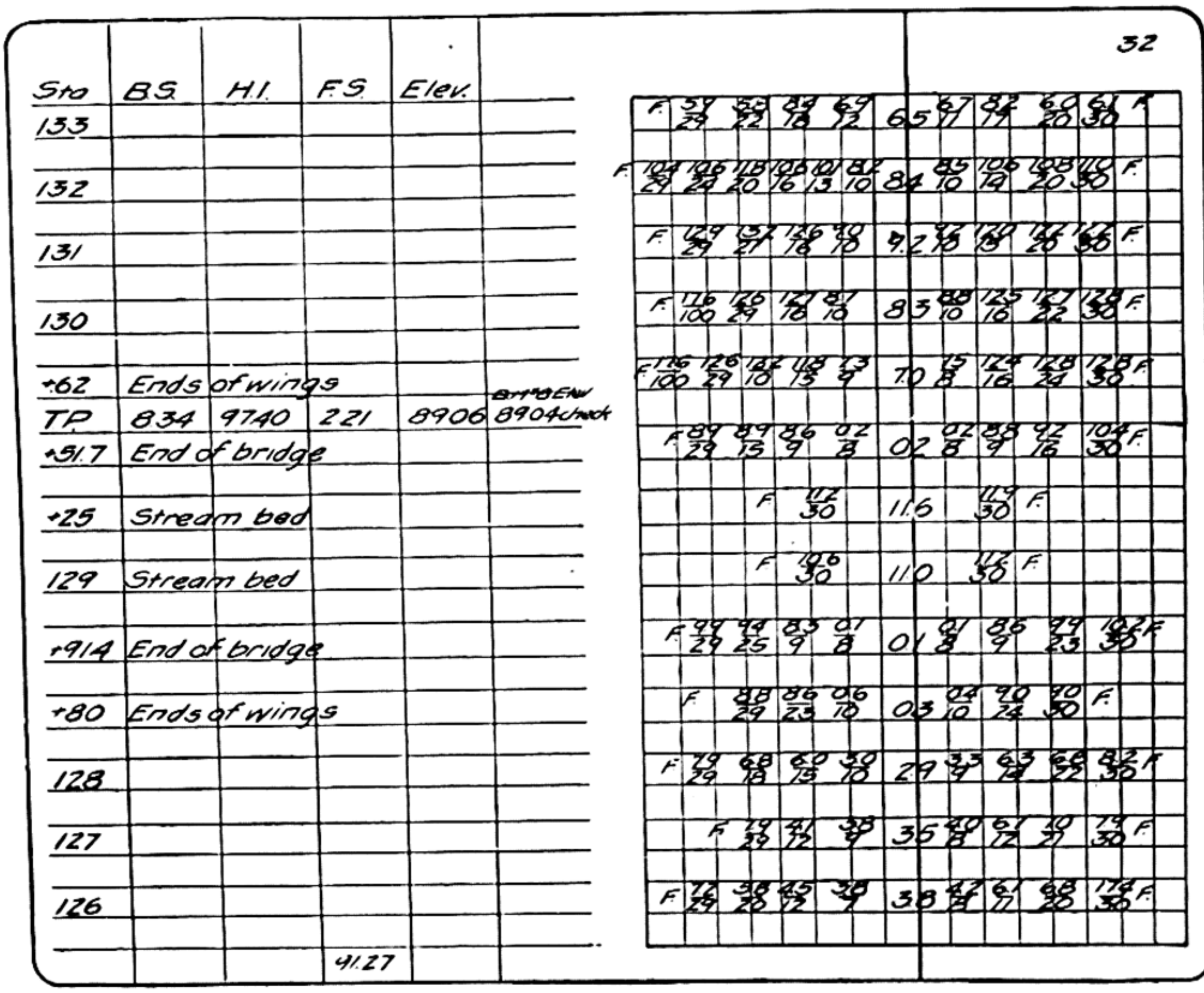

FIG. 11.—Typical level survey field notes.

NOTES.—When recording the bench-mark levels, it will be convenient to have the notes read from the top of the page downward. But the cross-section levels are more easily recorded from the bottom of the page upward so that "right" and "left" in the book are oriented with "right" and "left" on the ground. Mistakes in the use of "right" and "left" are not infrequent and when surveys run in the cardinal directions, it is perhaps wise to use "east" and "west," or "north" and "south." "Right" and "left" have advantages on diagonal or winding roads. Cross-section notes must not be crowded. A good arrangement is shown in Fig. 11.

Topographic Surveys for Bridges and Grade Separations.—Frequently, a special topographic survey will be required at sites of proposed bridges to procure adequate data for the designer. This is a function of the preliminary party, although it is some-

times done by a party with special equipment. Either the transit-stadia or plane table-stadia method will be suitable. Usually a large scale, such as 40 ft. to the inch, with 1-ft. contours, will be desirable.

This topographic map should be supplemented by the following information with reference to an existing bridge: type, condition and date of construction; length of each span, number of spans; adjacent bridges up and down stream; high- and low-water elevations; stream velocity; evidence of silting or scouring; surface elevations; stream depths at several places up and down stream; whether stream carries debris; danger from ice; complete cross-sections for abutments and piers; and borings at abutment and pier sites to ascertain underlying soil conditions.

Grade Separations.—A similar topographic survey will be needed at crossings where grade separation structures are in existence or proposed. An extensive survey is frequently necessary to determine the most advantageous location for a new structure. Where an under- or overcrossing has already been built, a sketch-in plan and elevation may suffice. Additional information will generally be needed as follows: kind of structure; clear width of roadway and whether the structure is skewed or perpendicular to the road center line; the angle between road and railroad center lines. For highways passing under the tracks the following information should be secured: elevation of highway and top of rails; vertical clearance; number of tracks; drainage conditions; borings to indicate soil conditions.

RIGHT-OF-WAY SURVEYS

A right-of-way survey with the accompanying right-of-way plat and conveyance should be a requisite on all permanent highway improvements. In states that do not have the rectangular system of land surveying, the right-of-way survey will parallel the road center line, just as a tract of land would be traversed. In such surveys, accurate descriptions of points of beginning and ending and all intermediate traverse points are of fundamental importance.

In states having the rectangular surveys, the roads usually follow the section lines. Such cases are more or less standardized as to the form of the description. They may depart materially from the standard when the road is relocated. Points of beginning and ending should be chosen with care and judgment.

Usually they will coincide with a land survey monument (section corner).

The plat which accompanies the description is fully as important as the description. It should show the data of the field survey to a suitable scale. The plat of an extensive project may be conveniently shown on a roll of paper, using a scale of 400 ft. to the inch. Parcels requiring special detailed treatment may be drawn to an exaggerated scale opposite their proper places on the small plan. Such an arrangement facilitates field examination.

These right-of-way maps should show the following:

1. The center line of the highway.
2. Right-of-way lines and actual width of right-of-way.
3. Ties to pertinent land lines and monuments.
4. Official designation of the land sections through which the highway passes.
5. Streams.
6. Bridges and culverts.
7. Railroad and trolley right-of-way.
8. Telephone and power lines.
9. Any distinctive object that will serve to fix and establish the right-of-way for future reference.

CONSTRUCTION SURVEYS

The surveys that are likely to be required during construction may be listed as follows:

1. Checking plans and staking the center line.
2. Cross-sectioning and setting slope stakes.
3. Setting finish stakes.
4. Setting pavement stakes.
5. Staking bridges and culverts.
6. Referencing land monuments and transit points.

The first step in the construction survey should be to check the plans to ascertain any changes that were made from the preliminary line. This is a field operation. While it is being done, the construction stakes should be set. Where the line follows a traveled road, each 100-ft. station may be marked on line by a spike driven into the ground through a red rag. Each of these spikes should be referenced by hubs set at a uniform offset distance, say 30 ft., on one or both sides of the road. New P.I.'s and P.T.'s will be set and referenced, as was done during the preliminary survey.

Cross-sections.—The work of the cross-section party should be kept well ahead of the construction. A set of cross-sections will be taken at the time that the slope stakes are being set. These cross-sections are to be compared with the original cross-sections of the preliminary survey and should be taken at the same places. These should be at each 100-ft. station and wherever the ground makes a noteworthy change in slope between stations. Special care should be taken to secure the elevations at the points where there is neither cut nor fill (called "0.0 points"). Three of these elevations will be encountered in going from cut to fill or from fill to cut, one on each side in the ditch, and one on the center line.

Slope Stakes.—In general, slope stakes driven vertically should be set for each station at the toe of the slope in fills and at the top of the back-slope in cuts. Slope stakes for deep cuts are often disturbed by construction operations and it is desirable, at times, to offset the stake about 3 ft. to avoid its being loosened. The slope stakes will generally be set with an ordinary level, although a hand level will be found sufficiently accurate under some conditions. The depth of cut or fill at the sides of the section and at the center line, and the distance to the center line should be marked on the face of the stake and the station number should be marked on the back of the stake.

Finishing Stakes.—As the contractor finishes his rough grading, the engineer will set finishing stakes. These should be set at each station, or oftener, and be given special identification, such as a blue mark on the top which will represent the elevation to which the contractor will finish his grading. In fills, the stakes should be set to provide for settlement which will be about $\frac{1}{10}$ ft. for each foot of fill. This does not mean that the fill is widened. The fill is raised at the shoulder lines but the width is held. When the fill finally settles, the side slopes will be as prescribed by the plans, although they are actually steeper when the contractor finishes his work. Usually the roadbed will be finished to a level cross-section, and if so, the finishing stakes in both cut and fill will be set on the shoulder lines at the elevation of the center-line grade.

Pavement Stakes.—Additional lines of stakes will be required for the construction of a pavement. The engineer and the contractor should agree upon the details of placing these stakes in order that delays and misunderstandings may be reduced to a

minimum. Usually, a line of stakes will be required on each side of the proposed pavement, set at a convenient offset distance from the edge, at edge, or at center grade. A convenient spacing is 50 ft. on tangents and straight grades and 25 ft. on vertical and horizontal curves.

Staking Bridges and Culverts.—Bridges and culverts must line up with the road. To assure correct alignment the engineer should take his line from transit points and should not rely on intermediate hubs that are supposed to be on line. Several transit points should be set on line near the bridge site and be

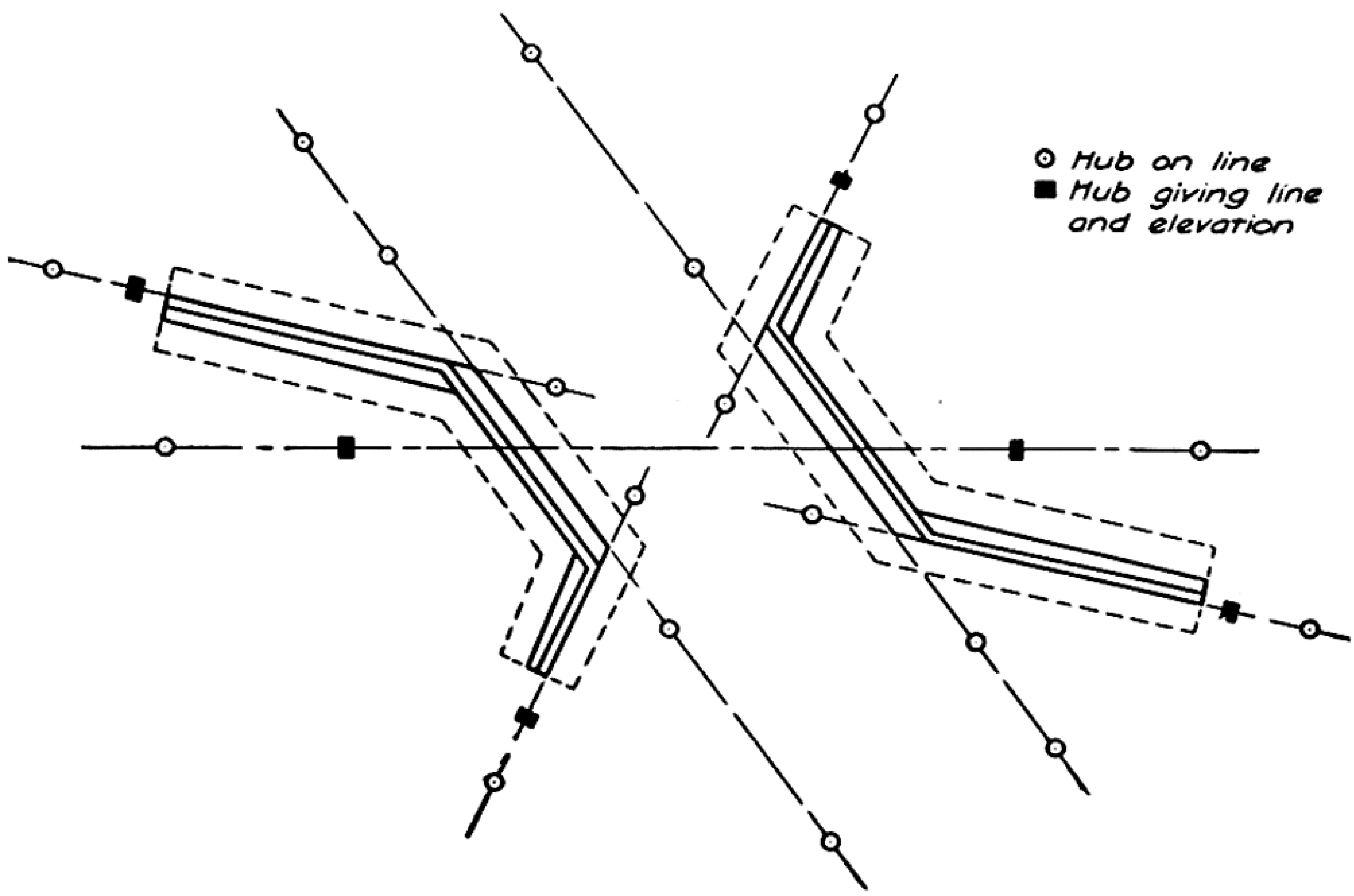

FIG. 12.—Illustrating a method of setting stakes for culvert construction.

carefully referenced for use during construction. Probably a piece of iron pipe, or iron bar, will be most satisfactory for that purpose. For small bridges and culverts one point on the center line on each side of the structure will suffice. These should be set outside the limits of construction operations. Greater care must be taken in providing reference points for large bridges that are likely to be under construction for some time. To make sure of a backsight at all times, two center-line points should be set on each side of the bridge, well beyond the limits of construction.

Bench marks for fixing the pier and abutment elevations are equally important. Permanence is the first requisite, but acces-

sibility should be considered when selecting the location of bench marks as they will be used many times with the instrument in various locations on the site.

Figure 12 illustrates a typical method of staking out a bridge. In setting the stakes the engineer should keep the needs of the contractor in mind. The latter is usually a practical and experienced man, but he must have enough stakes to fix the head walls, wing walls, and piers in the proper place and at the correct elevations.

Referencing Land Monuments and Transit Points.—During construction, everyone should be on the lookout for land monuments which may or may not have been found by the preliminary survey party. Each monument that is discovered should be carefully referenced so that it may be reset when the construction work is completed. In the case of an earth or gravel road the original stone or an iron bar may be reset in place and referenced permanently. With a pavement it will be necessary to drill a hole in the concrete above the stone or bar, and in it set a lead plug, a piece of pipe or an iron bar. The descriptions of monuments and references should be preserved most carefully and later recorded in the office of the proper official.

Transit points should be referenced and preserved in the same way, but they are not recorded. Usually all P.I.'s, P.O.T.'s, P.C.'s, and P.T.'s will be referenced. It is oftentimes desirable, for uniformity, to reference such points by four small concrete posts set so that lines connecting diagonally opposite posts will form an X with the transit point at the intersection of the two lines. These posts should be placed out of the way of mowers and graders, but in a conveniently accessible place. Probably a location near the fence line is most satisfactory.

FINAL ESTIMATE SURVEYS

All final payments on contracts should be based on a final survey. This would begin with taking cross-sections over the finished road at the same sections that were used for the first cross-sections. The difference between the two will give the *actual* cut or fill. Following this would come the measurement of all completed structures such as bridges, culverts, and the pavement itself. This can be done most satisfactorily by the resident engineer and the inspector.

As the engineer makes this final estimate survey, he should revise a set of construction plans to show the road as built. This can be done most readily by making proper notations on a clean set of blueprints. These notations should include changes in alignment and grade; station and description of each structure; land ties; bench marks; earthwork quantities; corrected locations of fences, buildings, landmarks, and the like.

II. ROAD PLANS

As might be expected, the plans prepared for road improvement vary in completeness and form among the various engineers and state departments. In general makeup they are much the same, consisting of a plan and a profile always, and frequently being accompanied by the cross-section for each station.

Profile.—A center-line profile is usually shown. It is sometimes the profile of the finished roadway, sometimes that of the foundation for the hard surfacing, and sometimes a "grade" profile or profile passing through the balance line of the cross-section. Probably the most common practice is to show the profile of the finished surface.

The scale to which the profile is drawn also varies greatly. A scale of 10 ft. per inch vertically and 100 ft. per inch horizontally, or of 200 ft. to the inch horizontally, is often used.

A horizontal scale of 40 ft. to the inch, and a vertical scale of either 8 ft. to the inch or 4 ft. to the inch, is also widely used and is probably the most satisfactory. Profiles drawn to this scale are voluminous and inconvenient to handle in the field, but are desirable from the standpoint of design.

The profile of the existing road surface and the established grade line are always shown and sometimes the profile of the ditches. The profile should also show all bridges and culverts, lines of tile, and catch-basins. The location of bench marks and their elevation is given, and the elevation of the established grade line is marked at every break in the grade line, and sometimes at every station. The name of the road, the date of the plan, and the scale are essential parts of the plan.

Plan View.—The plan view is usually on the same sheet as the profile and the horizontal scale is the same as the horizontal scale of the profile. The plan view is sometimes broken at turns or deflections and the center line maintained as a straight line so as to keep the plan on the sheet. On this view are shown the

FIG. 13.—Plan for contract road construction.

(Facing page 46)

existing features such as fences, culverts, bridges, houses, driveways, intersecting roads, and such other information as may assist in the design. The lines of the hard surface and the location of any bridges or culverts to be built are also shown.

Cross-sections.—The cross-sections are platted in computing the quantities of earth work and are of use in construction in showing where surplus material is found and where it is to be deposited. Usually, the area of each cross-section and the quantity of material between sections is indicated. The sections are platted to a scale of 10 ft. per inch, 8 ft. per inch, or 4 ft. per inch. The larger the scale the more accurately the cross-section areas can be obtained, but the sections become exceedingly voluminous if platted to a scale of 4 ft. to the inch. Figure 13 shows a typical road plan.

SURVEYS FOR CITY PAVING

The surveys made as a preliminary to city paving must be made with greater accuracy and detail than is necessary for road surveys because of higher property values, more exacting requirements as to connections to existing and proposed improvements, and more rigid requirements as to drainage.

Monuments.—As with highway surveys, one of the first steps should be the making of a thorough search for monuments. Here the engineer will find many perplexing and conflicting conditions that require very good judgment and considerable ingenuity for their satisfactory and equitable solution. Oftentimes, monuments that once existed at street intersections and on property lines cannot be found. In such a case the engineer must seek to harmonize existing building and sidewalk lines with the requirements of the plat. Frequently, a property line that is satisfactory in one block will not fit an adjacent block, although the street line is supposedly on a straight line. Vexing excesses and deficiencies in street measurements arise in many small towns and cities. The engineer must proceed with all of these things in mind and remember that his final location must depend upon good common sense and sound engineering principles.

General Method.—In many instances, one survey will suffice for both the preliminary and the construction survey. Methods of construction and differences in opposite curb elevations necessitate curb lines and elevations so that the center line is not needed as the reference line. Each curb line should be projected through

as long a distance as possible with due regard to existing and future paving and symmetry about the street center line.

The hubs should be set at 25-ft. intervals on an offset line 2 or 3 ft. back of the face of the proposed curb. On steep grades the interval might be increased to 50 ft. In the business district, where most of the roadway space will be taken up by the pavement, it will be convenient to cut small crosses on the sidewalk in lieu of the hubs.

The data of the preliminary survey should consist of everything that relates to existing structures and conditions. This would include the location and elevations of building entrances, sidewalks, driveways and alleys, poles and electroliers, hydrants, manholes, culverts, inlets, railroad and car lines, underground pipes with notations as to their condition, outlets, and the drainage area of the street.

Cross-sections.—Cross-sections should be taken in the usual manner at 50-ft. intervals. Elevations should be taken on the top of hubs to the nearest hundredth of a foot with particular care for check elevations both before and during construction. Bench marks should be frequent and permanent. A standard type of concrete bench mark is best, but lacking that, fire hydrants will be satisfactory. During the leveling, particular attention should be given to the elevation of intersecting streets, both paved and unpaved, as well as all alleys, driveway openings and sidewalks, to make sure that the designer will have sufficient data to lay out his part of a harmonious whole.

Oftentimes, it is convenient for reference and the computation of costs to survey each block as a unit; beginning anew at zero after crossing an intersection.

Stakes.—The stakes of the preliminary survey will be supplemented as construction proceeds. Additional stakes should be set for alley returns, for all curves if not completely staked during the preliminary work, and for tangent points and centers of curb arcs at intersections. Where "built up" forms are used for these small curves, hubs for the two tangent points will fix the location and the hub for the center of circle will not be needed. A grade stake will be needed to fix the elevation of the mid-point of the curve. Enough stakes should be set in the intersection to guarantee a finished surface that will conform to the plans.

When one set of stakes is used in this manner it will be convenient to furnish the foreman with a sheet giving cuts and fills

at each station. Setting the tops of the hubs to grade or marking them an even number of feet above or below grade, requires considerable time and is frequently impracticable. A good form setter can do satisfactory work from the indicated cuts and fills. When his string for the top of curb is stretched, errors in alignment and elevation can be discerned by eye. After the forms are set the engineer should make a second inspection by eye. This should suffice except in the cases of grades under 0.3 per cent, where an instrument should be used to check the forms to avoid water pockets.

PLANS FOR PAVING

Plat.—Usually a complete set of plans for a street improvement will include a plat of the city, or a portion of the city, with the proposed paving district and streets to be paved indicated thereon. Curb radii, catch basins, curb inlets, storm sewer lines, valley gutters, crossing plates, and similar special features may be shown most conveniently on the larger scale plan, which is placed above the profile on the standard plan-profile.

Profiles.—The plan and profile may be shown most conveniently on standard plan-profile paper or cloth. The plan, which is usually placed above the profile, should show all features that may be useful to the designer. The most convenient scale is 100 ft. to the inch.

The profile of the center line of the proposed pavement is sometimes useful and may be plotted. In laying the grade the engineer should make his design conform to the established grade, if there is one. If there is no established grade, he should try to fit his pavement to existing structures, side-walks, nter-secting streets, and the general lay of adjacent property.

Frequently, the elevations of opposite points on the two curbs will be different and the elevations of both must be shown. The two curb grade lines may be drawn on the plan if the difference in elevation permits. It will be desirable to show the two curb line elevations and the property line elevations at each station, at each change in grade, and at alley and street intersections. This can be done conveniently by setting down the elevations in a systematic way. For example, the North (East) property line elevation would be placed at the top; slightly below it would be placed the North (East) curb grade elevation. The middle section would have the center-line profile and curb grade lines.

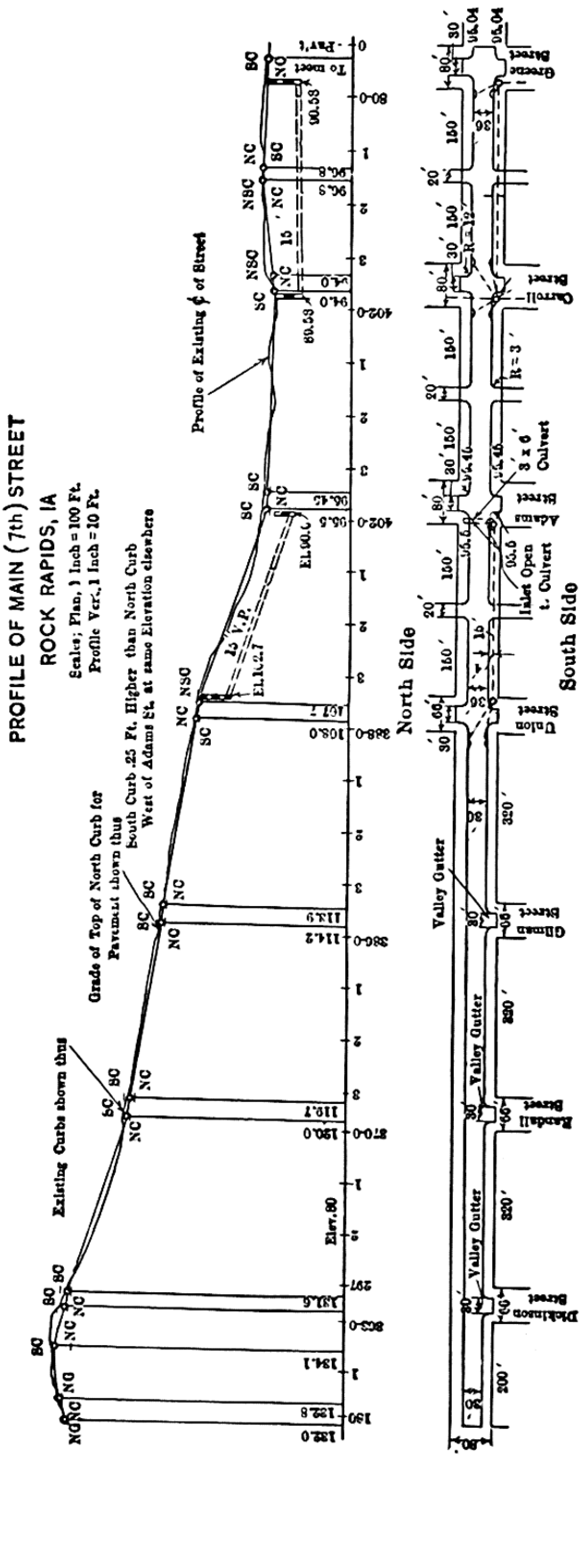

FIG. 14.— Plan for paving.

At the bottom would be the South (West) property line elevation; slightly above it the South (West) curb grade elevation.

The horizontal scale of the profile will be the same as that of the plan. The vertical scale will depend upon the flatness of the topography, but is generally 10 ft. to the inch, except where the topography is very flat.

If the gutter is not parallel to the top of the curb, as sometimes happens because of the requirements for drainage where flat grades are used, there should be a special note indicating the difference with its place of beginning and ending.

Cross-sections.—A sheet showing the details of the cross-section for the pavement will be included, but usually the cross-sections at each station are not included. These will be platted up for use in computing the quantity of earth-work but are not required for laying out the work. If several widths of pavement are included, a single typical cross-section may be shown, with a note indicating how the total crown will vary with the width of pavement.

General Information.—Generally several sheets of special details will be required in order to show the design of catch basins, curb inlets, manholes, crossing treatment at intersections, and similar minor details.

Problem

1. Have the Surveying Department assign as one problem the survey of a half mile of highway or a few blocks of a city street, the notes to be worked up and plotted in accordance with one of the standard methods of preparing road or street plans.

CHAPTER III

DRAINAGE AND CONTROL OF EROSION

The construction of highways ordinarily involves the installation of drainage and frequently includes provisions for the prevention or minimizing of erosion. Ordinarily the design of adequate drainage is comparatively simple, and the drainage works relatively inexpensive; likewise the provision for the control of erosion. Nevertheless these problems must be recognized, and in those cases where conditions require it elaborate installations for the accomplishment of these purposes must of necessity be provided.

DRAINAGE IN HUMID CLIMATES

The nature and extent of the drainage that must be provided will vary with the type of soil on which the road is built, the precipitation in the area where the road lies, and the topography. Erosion problems vary with the same conditions.

Types of Drainage Problem.—In areas where the annual precipitation is 25 in. or more, one or both of two types of drainage problems may be encountered in road construction. The one with which the road builder is most familiar involves the disposal of storm or snow water in so expeditious a manner that the percolation into the soil immediately under the road surface will be reduced to a minimum. This is ordinarily called *surface drainage*, and the standard solution is the simple one of providing open channels to carry the storm water from the proximity of the traveled part of the road. The second type of drainage problem is the prevention of the flow or percolation of water, other than that which is being carried in the ditches, into the soil in the upper 2 or 3 ft. of the subgrade under the road surface. This water, of course, originates in precipitation but sinks into the soil and collects in undersurface pockets of porous materials or flows in underground seams and porous layers, sometimes for great distances. It is usually referred to as "free" water to differentiate it from water held in the soil by surface tension or capillarity. Free water, which serves as an excellent

lubricant, is a prolific cause of landslips along the side slopes of cuts or embankments that have been made in grading the highway. It may also serve as a reservoir to supply water that seeps into the soil immediately under the roadway surface or is drawn in by capillary action, rendering the supporting subgrade unstable.

The usual method of preventing damage from underground water is to place tile drains at locations that will intercept the flow of water before it reaches the road proper and to cut off by means of tile or ditches water-carrying seams that may contribute to landslips along the highway. That is, the cure for this condition is tile underdrainage designed after a full knowledge of the exact location of the source and movement of the underground water.

Side Ditches.—The surface drainage of highways is usually accomplished by side ditches which flank the portion of the roadway that carries the traffic. In most instances a simple side ditch is all that is required, but sometimes special provisions are needed for the effective removal of a very large amount of storm water that may be expected during certain seasons of the year.

The side ditch may be V-shaped if the capacity does not need to be very great. In those areas where it is desirable to provide a trackway that can be used in summer by the vehicles serving the farms, particularly for agricultural machinery that ordinarily should not be drawn on a paved surface, the trapezoidal, or flat-bottom, type of ditch is sometimes employed with the expectation that during most of the farming season the ditch will be dry and can be used for traffic. This is common practice in many areas, but it seems preferable to many designers to provide, along the paved portion of the highway, shoulders that will be wide enough to take care of this kind of traffic. From the standpoint of the hydraulics of the problem there is no particular choice between the two types of ditch. Obviously if very large capacity is desired, the trapezoidal type will be used, since the capacity can be secured without the necessity of using a depth that will introduce side-slope problems. The various designs commonly employed for side ditches are shown in Figs. 35 and 36.

Run-off.—The amount of water to be carried by the side ditch along a highway is the run-off from the area contributing thereto.

Primarily this water comes from the portion of the road between the ditches, upon which there is a more or less impervious wearing surface, and from the earth shoulders flanking the roadway surface, which are usually fairly hard. It may be assumed that for storms of short duration, about 75 per cent of the water that falls on the area between side ditches will run to the ditches; and that for storms exceeding 15 minutes in duration, all of the water falling on this area will run to the ditches.

Some water will reach the road ditches from the areas between the ditches and the right-of-way lines and from adjacent land that slopes toward the ditches. The rate of run-off from this area will depend upon the slope, the character of the soil, and the nature of the growth thereon. Lands covered with growing crops or with brush and timber will absorb much of the water from storms that last an hour or more, except when the storms recur so frequently that a condition of saturation is maintained in the soil. In contrast, rocky or frozen soils will absorb very little water after the first few minutes, but there are very few locations in which the rocky soil does not carry some growth and some soil that will absorb water. Flat lands will deliver less runoff than those that are sloping. All these things may properly be taken into account in estimating the amount of water that will reach the side ditches. For general purposes it may be assumed that for storms of more than 40 minutes' duration and for the short downpours at very high rates all the water falling on the drainage area will reach the side ditch. Knowing the total area, the rainfall characteristics of the region, and the slope that may be obtained for the ditch, the area of the cross section of ditch may be computed by determining the velocity by means of the familiar Chezy formula, using Kutter's formula for determining C. For the usual highway condition, S in Kutter's formula may be assumed at 0.022. A careful analysis should be made to insure ample capacity where the side ditches must carry the water for long distances or where a considerable area adjacent to the highway drains to the road ditches. In this latter case, especially in hilly country, a surprisingly large volume of water may reach the highway.

Although under many conditions it is not necessary to go to the refinement of computing the ditch capacity, it is imperative to provide sufficient longitudinal slope to insure that the ditch empties quickly. Along the highways may be seen many

instances of ineffective surface drainage which may be attributed to failure to provide a continuous gradient sloping toward the outlet.

Supplementary Ditches.—It is often desirable to intercept, by means of a supplementary ditch roughly parallel to the road, water flowing from adjacent lands toward the road. Such water is frequently the cause of landslips in highway cuts.[1] A supplementary ditch is especially advantageous where the amount of drainage area to be provided against is large or the slope toward the road such as to make it difficult to control erosion. The slopes of cuts can be protected in this way by means of supplementary ditches 20 ft. or more back of the upper edge of the side slope. The use of supplementary ditches is illustrated in Fig. 36.

Tile to Supplement Side Ditches.—If side ditches on flat grades must carry storm water for long distances and in considerable quantities, it is to be expected that during rainy weather the ditches will impound water for a considerable period of time. This creates a condition favorable to the percolation of water into the subgrade in quantity sufficient to lower appreciably the supporting strength of the subgrade soil. To expedite the removal of the ditch water and thus minimize this hazard, tile drains are laid below the side ditches to supplement the ditch capacity. To be effective, such tile must be of ample size and have a free outlet so that full advantage may be taken of the capacity. Storm-water inlets must be provided, spaced a few hundred feet apart, to carry the water from the ditch to the tile. These may be in the form of small concrete catch basins or simply a vertical tile provided with a beehive type of grating which will not readily clog with weeds or other trash carried by the water. The so-called "blind" catch basin, which is merely a section of about 3 ft. of the trench backfilled with broken stone, broken tile, or gravel, will also serve the purpose.

The principal advantage of supplementary tile is realized when soil or accumulations of trash partially block the ditches, thus

[1] Downs, William S., "Earth Slip Hazards and Control," *Eng. News-Record*, Vol. 104, p. 794, May 15, 1930.

Kane, Wallace B., "Stabilizing a Slipping Fill on a Hillside Road," *Eng. News-Record*, Vol. 115, p. 184, Aug. 8, 1935.

Ladd, George E., "Landslides and Their Relation to Highways," *Public Roads*, Vol. 8, No. 1 p. 21, March, 1927.

preventing the free flow of the storm water along the ditch. It will escape through the inlets, rather slowly to be sure, and thus eliminate impounding in the ditches. In areas of considerable snowfall, there will usually be a period during the spring thaw when the ditches are only partially effective, and considerable water will be held in the ditch if tile is not used or if there are not adequate inlets to the tile. Generally the storm water will have passed through the tile before very much ground water from the same storm reaches a tile, and therefore the tile will also serve as underdrainage to lower the ground-water level, at least to some extent. In continued wet weather the efficiency of the tile in lowering ground water will be greatly reduced by the fact that it is loaded with surface water delivered through the inlets.

Drainage of Embankments.—The surface drainage of a road surface located on an embankment is not much of a problem when the fill is not more than about 6 ft. high and composed of carefully compacted soil (page 158). The low fills can be built with reasonably flat side slopes which, in humid areas, will soon be covered with grass which will protect the slopes from erosion. When the embankment has to be built with sandy or other erosive soil, and in any case if the embankment is more than about 6 ft. high where of necessity the side slopes must be relatively steep, the maintenance of the side slope becomes a troublesome problem. Storm water that flows down the side slopes of the fill will carry away great quantities of soil unless preventive measures are taken. This type of erosion may be minimized by constructing a ridge or low curb along the edge of the roadway to confine the water that falls on the top of the embankment and carry it to paved flumes arranged to carry the water down the side slopes at appropriate intervals. If a paved road surface with a marginal curb is used on an embankment, the shoulder is graded to slope toward the pavement, and the water is carried on the pavement to flumes down which it flows to the foot of the embankment. Curbs of light metal, held by stakes, may be used on unpaved road surfaces to carry the storm water to metal flumes extending down the embankments. In this way the only water that will flow down the side slope will be that which falls on the slope.

Underdrainage.—Storm water that percolates into the soil flows downward and laterally, its course depending upon a great many conditions of soil and topography. The distance from

the surface of the ground to the level at which free water is found in the soil varies greatly with topography and soil texture. Underground water flowing freely through the soil may be the cause of instability in the soil under a road surface and very often is the underlying reason for the failure of the wearing surface to withstand the pounding of traffic. The level of free underground water may be held to such a depth that it will not jeopardize the stability of the soil supporting the road surface if properly designed tile drains are employed. It must be recognized, however, that although tile drains will remove the free water quickly from the granular, or porous, types of soil, such drains are not effective with the fine-grained and waxy types of soils (page 87). Many types of soils, and especially fine-grained silts and clays, retain moisture in the form of a film enclosing each particle and perhaps some free water held in the minute space between the grains of soil, which gives the soil a gel-like consistency of varying degrees of instability according to the texture of the soil and the moisture content (Chap. IV). Such moisture is held quite tenaciously through capillarity and surface tension and will not respond to underdrainage. However, underdrainage may reduce the supply of free water from which capillary moisture is replenished as it is dissipated by evaporation. Evaporation is slow, and no great change in capillary moisture takes place from month to month through the seasons.

It will be convenient to consider three distinct conditions that are encountered in designing tile underdrains for highways.

The first condition is one already mentioned in which the tile is employed to supplement the surface drainage in addition to lowering the ground-water level. For tile drains of this character the safe rule is to provide sufficient capacity for both purposes on the assumption that water is being delivered to the tile from both sources simultaneously. That will ordinarily mean a tile of at least 8 in. in diameter.

The second condition is that which is encountered in relatively flat country where the water must be carried for long distances to reach an outlet, and where the soil is of such a type that it reaches a state of saturation rather quickly after a storm. In these instances the generally accepted principles of land drainage may be employed in determining the tile drainage system if the soil is of a texture that will respond to tile drainage. Usually it is desirable to place two lines of tile along the road

at a depth sufficient to lower the ground-water level to at least 4 ft. below the road surface. On the basis usually adopted for land drainage, one line of tile is adequate to drain the right-of-way of a road 4 rd. wide, but experience has shown that this is not generally adequate for the relatively impervious area to be drained on a highway and that two lines of tile are needed (Fig. 36).

The third condition is one that is encountered in hilly country more frequently than elsewhere but may exist under any topographical condition. This particular condition is manifested by seepages of water in the most unexpected places and by partial saturation of the road surface in locations remote from accumulations of surface water. These seepages are evidences of the flow of underground water that has been following along on top of dense strata or in veins or porous layers in the soil and has finally reached the surface of the ground or near enough to the surface to affect its stability. This condition results from the flow of free water in the soil as previously described and is most common in hilly country although sometimes noted even in prairie country.[1] Where the layers of the several types of soil are nearly horizontal and of considerable thickness, it is comparatively easy to predict the behavior of underground water. When the soil consists of pockets or of layers of materials of varying degrees of porosity, lying at various angles with the horizontal and more or less faulted, the behavior of underground water must be carefully investigated. This condition exists not only where ledge rock, shale, or dense clays underlie the surface of the soil but also where the more porous clays, gravelly soils, or sandy loams predominate.

The drainage requirements of a road where such conditions exist will be exceedingly variable and can be determined only by a careful survey prior to beginning construction work and during the process of construction,[2] using test-hole[3] exploration wherever ground water is a probability. In regions where the ground does not freeze, examination is most advantageously made during the

[1] AYRES, QUINCY C., "A Study of Unusual Earth Road Conditions in Northeastern Iowa," *Public Roads*, Vol. 7, No. 3, p. 59, May, 1926.

[2] HOGENTOGLER, C. A., and E. A. WILLIS, "Stabilization of Muck and Sand Fill," *Public Roads*, Vol. 13, No. 4, p. 57, June, 1932.

[3] A soil auger is convenient for this; or if one is not available, a post-hole auger may be used.

season of maximum precipitation and immediately subsequent thereto. The purpose of such examination is to locate those sections of the road which appear to be affected by underground water; and when any are found, the exact nature of the flow of underground water can be determined by test holes. Such explorations are advisable where there are side-hill cuts, cuts at the summit of hills, and sections of road that lie somewhat lower than the land alongside.

The drainage method applicable to all these locations is to provide tile lines to intercept the flow of water at a depth 5 or 6 ft. below the surface of the road and to carry the water to an outlet remote from the road surface. The exact arrangement of the drains will vary with conditions uncovered by test-hole exploration and the topography. It is a wise precaution, where any seepage is noted in cuts, to lay the tile along the toe of the side slope with the trench backfilled with porous material to facilitate the entrance of water to the tile. The size of tile to use in such cases is largely a matter of judgment, as there is no reliable basis for computing it. Obviously it would be unnecessary to lay tile when the road surface is built on the porous soils such as sand and on the very sandy loams.

Subgrade Drainage.—The purpose of subgrade drainage is to minimize the effect of water upon the load-supporting capacity of the material upon which the roadway surface is constructed. It is important in this connection to differentiate between free water and water that is held in the soil by capillarity. Free water being in motion, the quantity in the pores of the soil at any time depends upon the texture of the soil (void size and volume), precipitation, and topography and will vary from an amount that will produce complete saturation to a quantity so small as to be negligible. The amount of water held in the soil by capillarity will depend upon the texture of the soil, the supply of free water available for replenishing the evaporation loss, and the extent of evaporation.

Suitable underdrainage systems will minimize the deleterious effects of free water but no means have as yet been devised for eliminating capillary water, and in humid regions the soil under a road crust is usually moist, even in dry weather. Since the subgrade is insulated by the road crust, there is little evaporation, and the quantity of capillary moisture in the subgrade soil will change but slowly during continued dry weather. On the other

hand, free water will be introduced into the subgrade during prolonged periods of precipitation through the cracks and joints in the road surface and from the shoulders alongside the surfaced strip. If the soil is of a type that swells when wet, this promiscuous wetting in small areas will cause unevenness in the road surface. Even concrete slabs will be lifted by the swelling of the soil. Types of soil that do not swell will be softened sufficiently to affect the supporting strength of the areas reached by this free water. Underdrainage does not aid this situation. Good maintenance of cracks and joints will minimize the infiltration of water.

There is considerable evidence to indicate that even with the most elaborate drainage provisions there will be periods when certain areas of the subgrade under the road surface will contain so much water that it will have low supporting power. The investigations of this subject indicate that there is enough capillary water held in the soil even in dry weather to preclude the possibility of ever achieving a truly dry subgrade, except with very porous soil. Many types of soil retain the water with great tenacity. It is this feature of certain types of subgrade soils that has led to the various attempts at stabilizing subgrades which will be discussed in another place.

Relation of Drainage to Frost Action.—The surface manifestation of the disturbance of a road surface from the expansion of freezing water in the supporting soil is frequently an irregular dome-shaped bulge which becomes unstable as soon as the supporting ice melts. From its characteristic appearance this has been dubbed a "frost boil." The effect of the freezing of water in the soil beneath a road surface may be confined to an area of a few square yards, or it may affect many linear feet of the road. A road surface of any flexible type on top of or adjacent to a frost boil is rendered unstable and often impassable when the ice melts, and even heavy wearing surfaces are seriously damaged in some cases.

The process of ice formation in the soil has been investigated by several laboratories and has been studied extensively in the field. The conclusions reached as to the nature and mechanics of the formation of the lenses in the soil point to drainage as one means of minimizing the damage.[1] The whole mass of the soil

[1] MULLIS, IRA B., "Illustrations of Frost and Ice Phenomena," *Public*

subject to freezing temperature expands when ice crystals form in the voids in the soil. It is obvious that, if these voids are full of water before freezing, the volume of the soil mass affected must increase when the water freezes. Such evidence as is available indicates that fine-grained soils, which hold water very tenaciously, are peculiarly subject to this type of expansion from the formation of ice. In other cases the water appears to squeeze out of the voids in the soil and collect in lenses of ice of which there may be several in the upper 2 or 3 ft. of the subgrade soil. The precise physical condition that favors the formation of ice lenses is not too well understood. It is known that after freezing starts, water will continue to move to the freezing zone through capillary action if there is a supply of water available. Ice lenses will also form in porous soils naturally free of capillary water if there is a drainage condition that permits free water to flow into the soil by natural gravitational action. These places would be unstable even in dry weather, as has been explained in the discussion of underdrainage.

So far as is now known, two methods are available for minimizing the disturbance of road surfaces from the expansion of freezing water in the supporting soil. The first is to provide drainage that will prevent free water under a slight hydrostatic head flowing under the road in seams and then rising in a subgrade. This is effective when the subgrade is composed of porous soils that will naturally respond to underdrainage. Some beneficial effects will be secured by draining the free water out of any layers or pockets of porous materials that underlie the fine-grained clays and provide a reservoir of water that is available to be carried up into the clay by capillary action. Such conditions are not common. The second method is one that must be resorted to when the subgrade is composed of the types of soils (page 90) from which the water cannot be removed by underdrainage. It involves removing the soil and replacing it with

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Roads, Vol. 11, No. 4, p. 61, June, 1930.

Taber, Stephen, "Freezing and Thawing of Soils as Factors in the Destruction of Road Surfaces," *Public Roads*, Vol. 11, No. 5, p. 113, August, 1930; "Frost Heaving," *Jour. Geol.*, Vol. 37, p. 428, 1929.

Benkelman, A. C., and F. R. Olmstead, "A New Theory of Frost Heaving," *Proc.*, 11th Ann. Meeting, Highway Research Board, December, 1931, p. 152.

more stable material or otherwise stabilizing the material by one of the methods discussed in Chap. IV.

DRAINAGE IN ARID AND SEMIARID REGIONS

In arid and semiarid regions the problem of drainage is primarily one of controlling surface water. There is seldom need for underdrainage along the highways in such climates, although seepages may occasionally exist in the most unexpected places, particularly in mountainous districts.

Drainage Methods.—In arid and semiarid regions the annual rainfall may be 15 in. or less, but the rain that falls may come during a very short season. Frequently the run-off in such areas is relatively high, and flood conditions may prevail for short periods of time. The ditches required to accommodate properly the flood flow may be of as great capacity as those used in humid regions. In fact, the side ditches must frequently be considerably larger than would ordinarily be employed in regions where the rainfall is distributed throughout the year. It is something of a shock for the uninitiated to drive along the highways in some of the dry western states and find elaborate drainage ditches with weirs, flumes, and other erosion-controlling devices in a ditch that is as dry as powder. At infrequent intervals, however, these ditches carry raging floods. The prospective run-off in such regions is exceedingly difficult to predict, and it is only by long familiarity with the rainfall and run-off characteristics of the area that one can become competent to design the drainage appurtenances for a road. One of the principal problems of drainage in such areas is that of preventing erosion, which will be discussed in another place.

DRAINAGE CONDUITS

Drainage conduits are concrete or metal pipe, burned clay or cement concrete tile, and monolithic concrete culvert barrels used for the hydraulic traffic carried by the drainage system. There are so many technical aspects to the design of these units that it has almost become a special field of drainage practice. Only the basic principles of application to highways will be discussed herein.

Drain Tile.—The tile employed for road drainage is the ordinary standard tile used also for land drainage. Tile of acceptable

quality for ditches of normal depth (about 4 ft.) will be assured if they comply with the standard specifications for drain tile of the American Society for Testing Materials (A.S.T.M. Designation C4-24). The tile that is delivered should be checked for strength and soundness just as is customary for any other construction material.

The ordinary maximum loads on drain tile are given in Table IV. For conditions outside those given in the table special computations are required to determine the load.

If drains are laid where there is some likelihood of soil movements displacing the sections of pipe, bell-and-spigot sewer pipe may be employed, but the joints are not cemented as would be the case in sewer construction.

Where lines of tile pass under traffic ways, the portion of the drain under the roads should be made of units strong enough to withstand the traffic load. In some cases it is desirable to use metal pipe rather than clay or concrete tile for the portion of the drain subjected to traffic loads.

If storm-water inlets are employed along the line of tile, these are commonly constructed by inserting a sewer pipe tee in the tile line and then placing a sewer pipe riser up to the surface. A beehive grating is placed in the bell end of the top section of the riser. Other types of inlets are also employed which are especially designed for particular locations.

The pipe for drains should be carefully laid on a bed shaped to fit the tile, and the backfilling should be carefully tamped at the sides of the pipe and up to the level of the top of the pipe. The remainder of the ditch may be backfilled by machinery.

The drainage of airports and similar extensive level areas subjected to traffic is a troublesome problem requiring a network of underdrains with inlets to carry the surface water to the drains. Perforated flexible metal pipe have been used for this purpose, as they can serve not only as storm-water drains but also to lower the ground-water level and resist the shock of surface loads to such an extent as to permit their being placed with safety in relatively shallow trenches.

Culverts.—The term *culvert* is used by engineers to designate the structures employed to carry water through highway embankments or under a roadway surface. In this treatise the discussion of culverts will be limited to those drainage channels which are entirely buried in the embankment. Short-span bridges are

designated as culverts by many highway departments, but such structures are outside this discussion.

The culvert is an integral part of the drainage system for the highway and is fitted into the cross-section in such a manner as to permit continuity of grade and alignment. Its function is to carry the water from one side of the traveled way to the other, and the controlling factors in the design of culverts are (1) provision of sufficient area of waterway to accommodate the flow of water through the culvert; (2) structural strength sufficient to carry the weight of fill and of the traffic loads that pass over the culvert. Ordinarily the culvert is circular or square in cross-sectional areas; but where headroom is limited, a rectangular cross-section may be employed in which the height of the opening is much less than the width. Under similar circumstances of restricted headroom a battery of circular culverts may be employed instead of a single circular barrel.

Size of Waterway.—The cross-sectional area of the culvert barrel is designed on the basis of the estimated quantity of water that must flow through the culvert at times of maximum hydraulic load. The size to employ in a given situation is best determined by estimating the drainage area contributory to the culvert, taking into consideration the factors of run-off that have already been discussed and then applying one of the well-known formulas to determine the area of waterway. Those who are familiar with the run-off characteristics in an area usually are able to select with considerable certainty the constants necessary to make these formulas fit the run-off characteristics of the region. One of the most widely used of these formulas for waterways is Talbot's formula, and Table V has been prepared from that formula, applying the coefficient recommended by the U.S. Public Roads Administration. This table may be used with considerable assurance for the determination of the waterways for highway bridges and culverts. There is given in Table VI a similar table, suggested by the American Railway Engineering and Maintenance of Way Association, which is particularly useful in areas where steep slopes prevail. It will be noted that Talbot's formula appears at the beginning of Table V.

Types of Culverts.—Culverts are constructed of a number of kinds of materials and in several forms. Any one of these may be used according to convenience and the cost of construction in a given location. It should be made certain, however, that

the structural strength of the particular type selected is adequate for the load it will be called upon to sustain in that location.

Reinforced Concrete Culverts.—Reinforced concrete is a desirable and widely used material for culverts because the form of the culvert can be adapted to the site and because the structure is monolithic and consequently particularly satisfactory for culverts where the rectangular section is desired and for culverts that pass through side-hill fills on considerable slope or for culverts that are built with drop inlets.

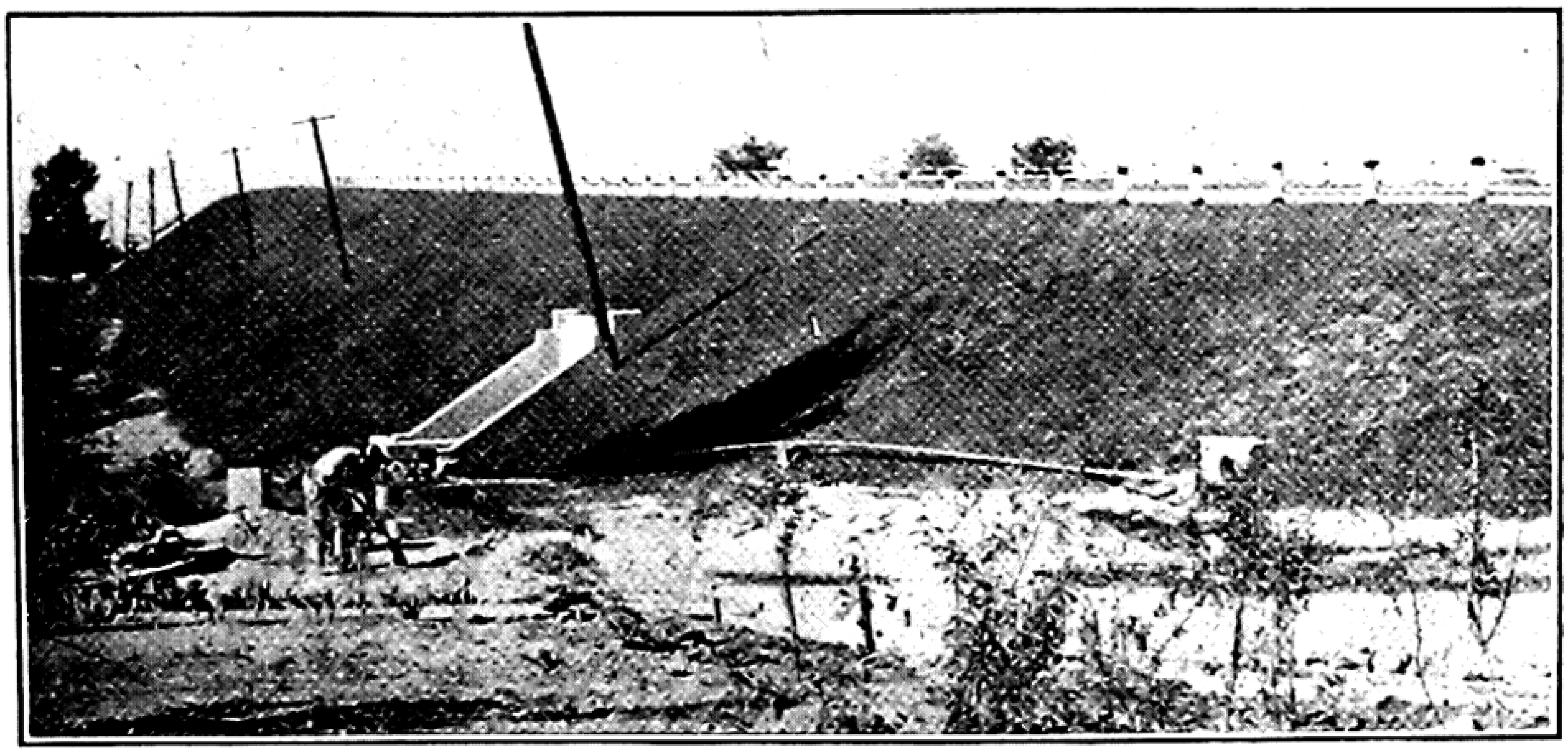

Fig. 15.—Culverts suitable for high embankments.

Materials for this type of culvert are obtainable in almost every locality and can be transported to the site at low cost because they are carried in small units. The strength of the concrete culvert can be adapted readily to the location and load to which it will be subjected. Figure 15 shows a few typical culverts, and Fig. 16 shows the drop inlet type.

Pipe Culverts.—Culverts are made of clay, cast iron, concrete, or sheet-metal pipe, fabricated into culvert barrels of various sizes and lengths. Figure 17 shows a few types of pipe culverts.

Loads on Culverts under Fills.—The load that a culvert must support consists of the portion of the weight of the embankment carried by the culvert barrel and a part of the weight of the traffic on the road surface over the culvert.

Experiments[1] have shown that the portion of the weight of the embankment that is carried by the culvert depends upon the

[1] Marston, Anson, "Theory of Loads on Closed Conduits," *Bull.* 96, *Iowa Engr. Exp. Sta.*, Vol. 28, No. 38, 1930.

Spangler, M. G., "The Supporting Strength of Rigid Pipe Culverts," *Bull.* 112, *Iowa Engr. Exp. Sta.*, p. 54, 1933.

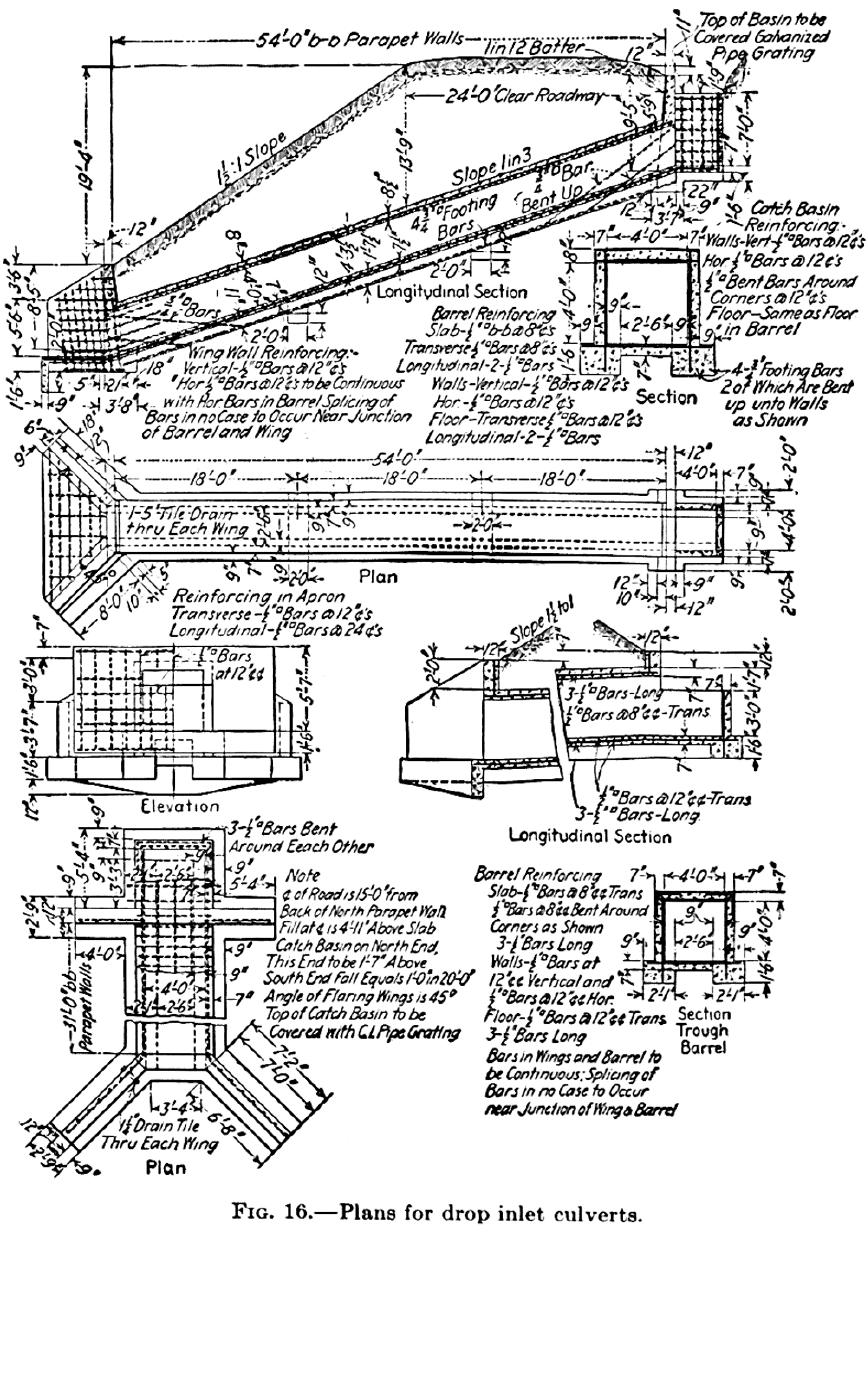

FIG. 16.—Plans for drop inlet culverts.

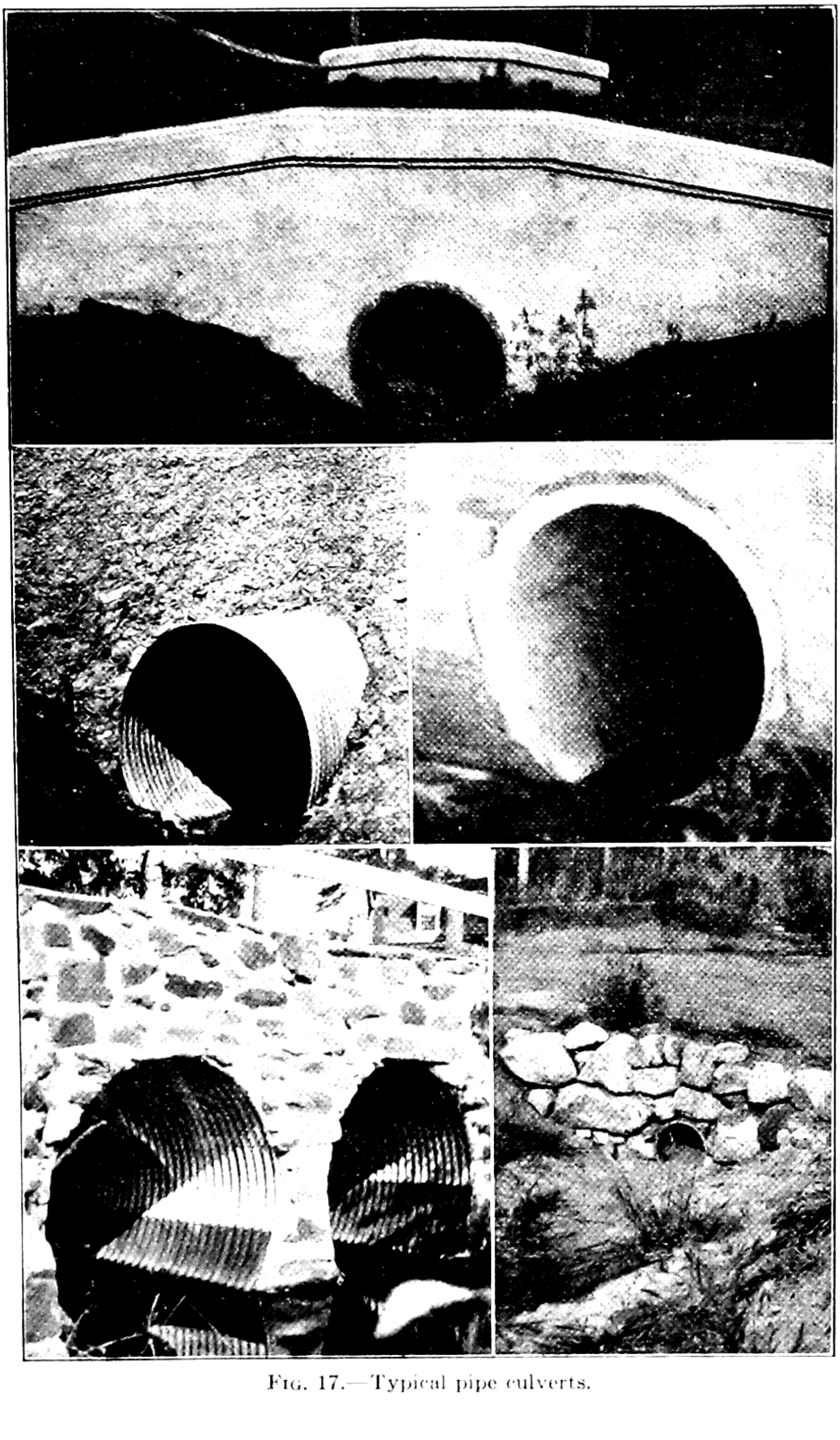

FIG. 17.—Typical pipe culverts.

method of placing the fill, the method of installing the culvert, the nature of the soil that supports it, and the unit weight of the fill material. The process of estimating the loads on conduits

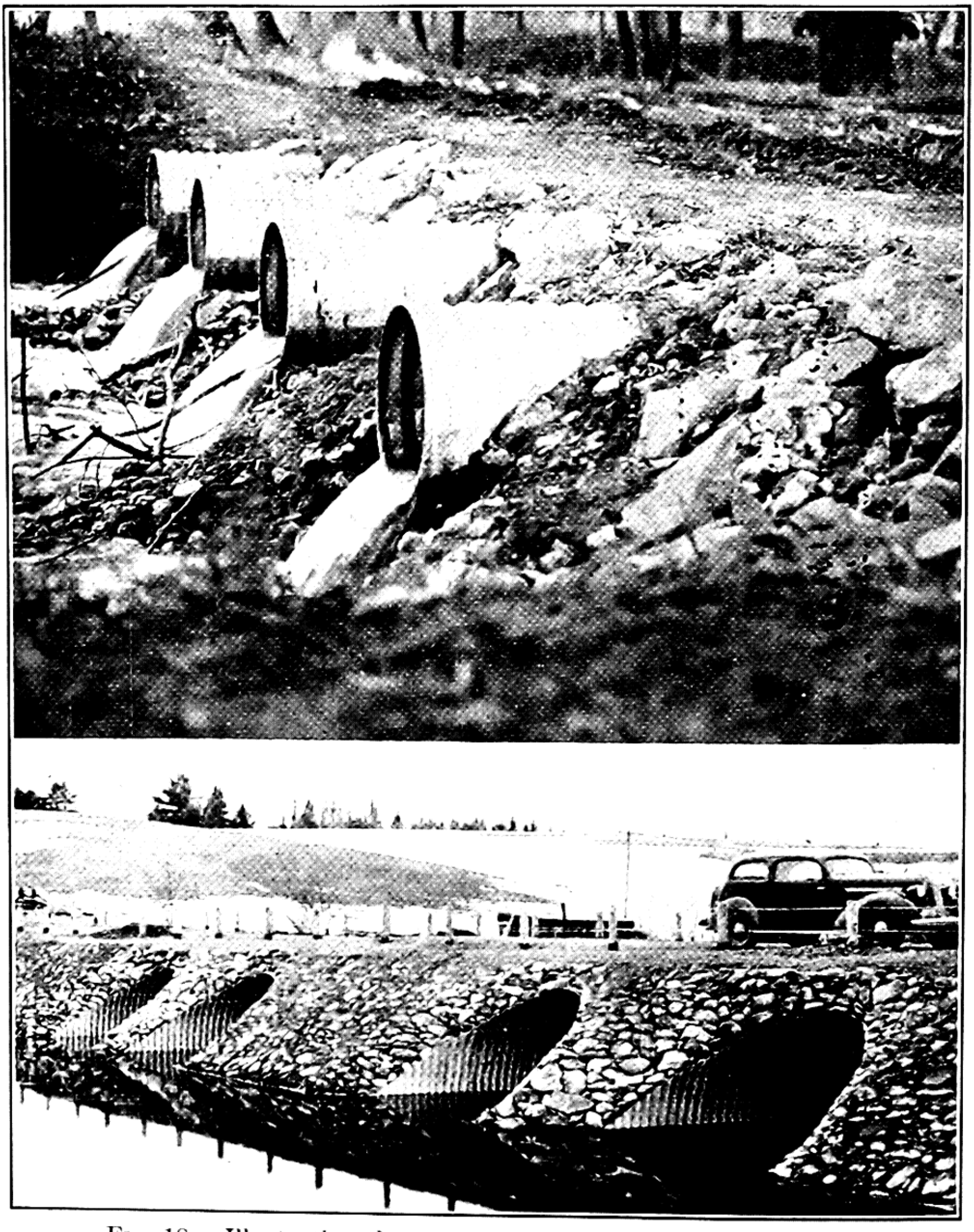

FIG. 18.—Illustrating the use of batteries of pipe under low fills.

under fills is too long for presentation herein but is set forth in the bulletin referred to above. Suffice it to say that the load due to the fill material may vary from the weight of a prism of fill directly over the conduit to a load equal to three times that amount. The probable maximum load can be determined in

advance for a particular location and method of laying when all the necessary data can be secured.

Behavior of Culverts in Ditches.—Sometimes culverts are installed by excavating a trench across an existing embankment and placing the culvert in the ditch which is later backfilled with loose material. Perhaps the embankment itself is increased in height by the addition of loose material deposited in the customary manner of grading. If the trench is of a depth greater than the height of the culvert barrel and not much wider than the barrel, the load is always less than the weight of the prism of fill above the barrel of the culvert. The theory of such loads has been established by numerous experiments, which are also applicable to pipe and tile used in the standard drainage system.[1]

Effect of Traffic Loads.—A part of the weight of the traffic that passes over the road surface is transmitted through the embankment to the culvert, and experiments have verified the applicability of the Boussinesq[2] theory of pressure distribution to this problem. Although the computations are tedious, they are simplified for routine application by the use of diagrams from which the proper computation factor may be taken for use in routine design problems. The extent to which traffic loads are transmitted to the culvert barrel depends upon the nature of the surface of the road surface. For unsurfaced roads or light gravel- or oil-mixed surfaces it is best to assume that the road surface does not effect any distribution of the load. If a wearing surface is employed that is relatively stiff, like a concrete or heavy bituminous macadam, the portion of the traffic load that reaches the smaller culverts, say up to 6 ft. wide, is much less than for the case of the unsurfaced road. Although no dependable data are at hand to indicate the extent of the reduction of loads on culverts through the load-spreading action of stiff wearing surfaces, it is safe to estimate a 50 per cent reduction.

[1] MARSTON, ANSON, W. J. SCHLICK, and H. F. CLEMMER, "The Supporting Strength of Sewer Pipe in Ditches," *Bull.* 47, *Iowa Engr. Exp. Sta.*, Oct. 10, 1917.

SCHLICK, W. J., "Supporting Strength of Drain Tile," *Bull.* 57, *Iowa Engr. Exp. Sta.*, Apr. 14, 1920.

Additional data on the subject will be found in the following bulletins of the foregoing Experiment Station, Nos. 82, 94, 96, 108.

[2] SPANGLER, M. G., CLYDE MASON, and ROBLEY WINFREY, "Experimental Determination of Static and Impact Loads Transmitted to Culverts," *Bull.* 79, *Iowa Engr. Exp. Sta.*, 1926. See also *Bull.* 96.

Supporting Strength of Culvert Pipe.—The capacity of a culvert pipe of circular cross-section to carry the vertical external load from the fill and traffic loads is determined by certain tests that have been adopted by the American Society for Testing

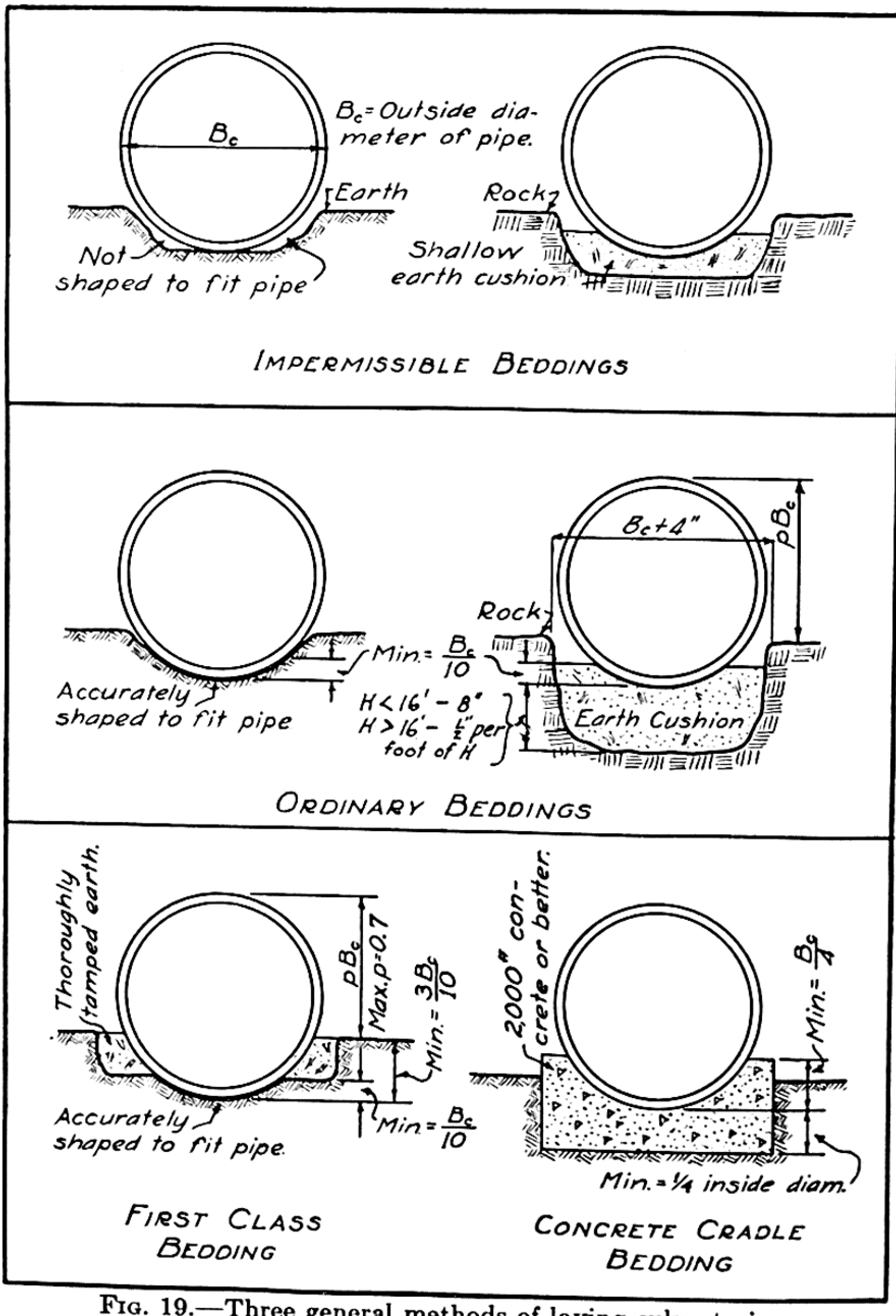

FIG. 19.—Three general methods of laying culvert pipe.

Materials of which the three-edge bearing test is the one most convenient to use.[1] The strength of the pipe as so determined

[1] Included in Standard Specifications for Cement-concrete Sewer Pipe, A.S.T.M. Designation: C14-24, A.S.T.M. Standard Specifications, 1933, p. 207.

multiplied by $1\frac{1}{2}$ is usually taken as the capacity to support external vertical loads. This is calculated in pounds per foot of length of pipe.

Culverts of the monolithic type with a rectangular cross-section of barrel are usually made of concrete and are designed for the required strength according to the standard practice in reinforced concrete design.

When cylindrical culverts are used, it is an economy to place the pipe on a carefully shaped bed and to backfill and tamp carefully so as to give support on the lower half of the barrel. Three general methods[1] of laying are shown in Fig. 19, and the load-carrying capacity of pipe laid by "first-class" methods is about 25 per cent greater than when laid by "ordinary" methods. The load-carrying capacity of pipe laid by "impermissible" methods is about 20 per cent less than when laid by first-class methods. The use of a suitable concrete cradle for bedding the pipe increases the load-carrying capacity to about double that secured by ordinary methods of laying. These factors are summarized in Table III.

TABLE III.—LOAD FACTORS BASED ON LAYING CONDITIONS EMPLOYED FOR PIPE CULVERTS

To determine the allowable external vertical load on a culvert pipe, multiply the strength at 0.01-in. crack as determined by the three-edge bearing method by the factor given in the following table for the laying method adopted.

	Laying method			
	Impermissible	Ordinary	First class	Concrete cradle
Factor	1.2	1.5	1.85	2.5

Adapting Culverts to Topography.—The prevention of erosion at the ends of culverts and in the streambed above and below the culvert can be accomplished by adapting the design of the culvert to the particular conditions that prevail at the site. The usual design consists of a barrel laid with a slight longitudinal slope in the direction of flow and at the elevation of the flow line of the drainage channel to be carried through the embankment. This is the normal and predominating design.

[1] SPANGLER, *op. cit.*, p. 15.

If the drainage channel to be accommodated has eroded to considerable depth below the surrounding terrain, and the embankment is 10 ft. or more in height, the culvert can be designed to aid control of erosion in the stream above the culvert. This is accomplished by placing a drop inlet on the intake end of the barrel. The channel will soon fill to the level of the inlet, and erosion of the streambed will cease for some distance above

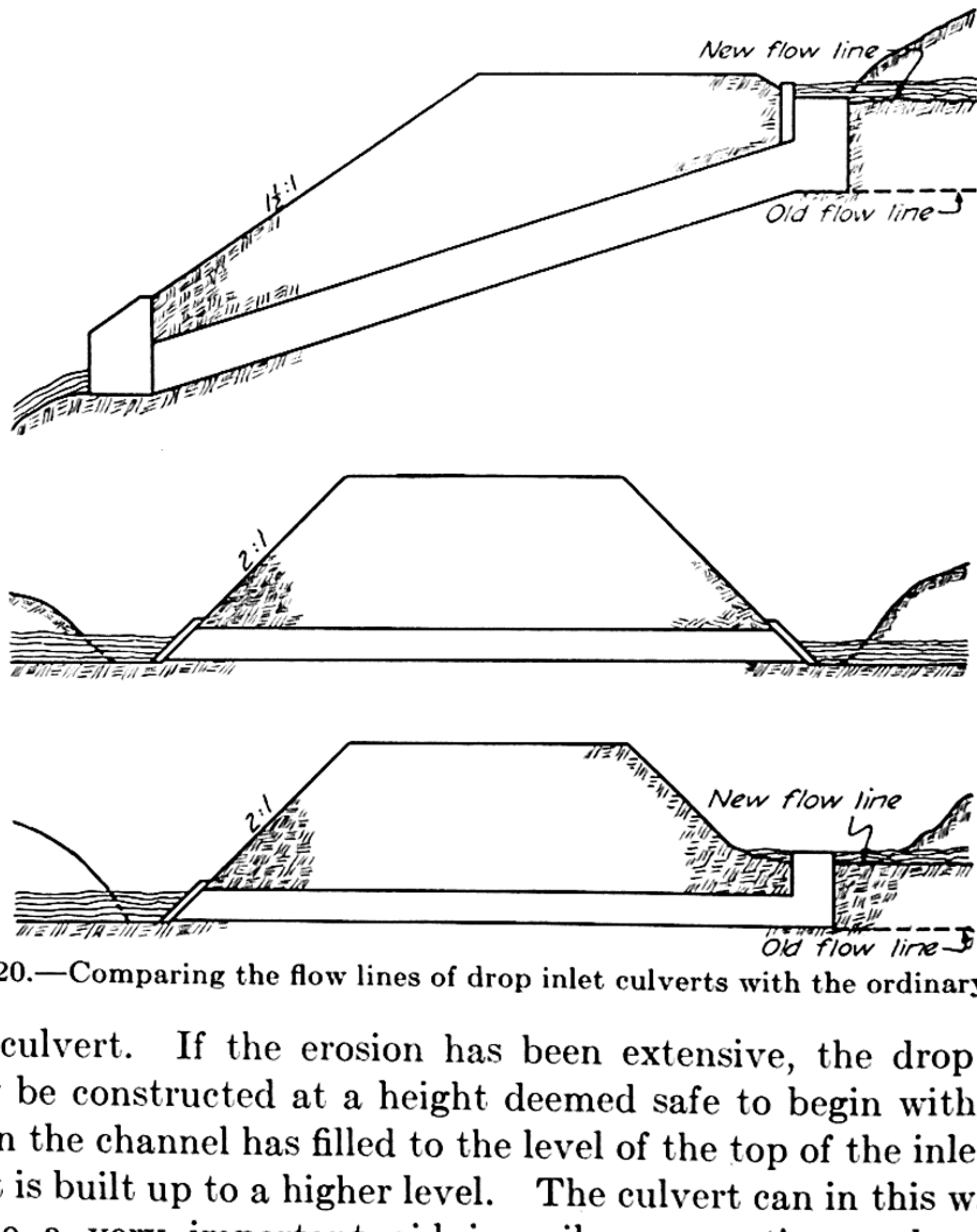

FIG. 20.—Comparing the flow lines of drop inlet culverts with the ordinary type.

the culvert. If the erosion has been extensive, the drop inlet may be constructed at a height deemed safe to begin with; and when the channel has filled to the level of the top of the inlet, the inlet is built up to a higher level. The culvert can in this way be made a very important aid in soil conservation and erosion control.

Frequently culverts must carry drainage through side-hill fills where the slope of the existing drainage channel is considerable. Here the culvert barrel may be laid on the slope of the existing channel with a short horizontal section at each end to minimize erosion near the inlet and outlet. Often it is desirable to use drop inlets on such culverts.

The several designs that may be employed for culverts are shown in outline in Fig. 20. It should be the aim to set the culvert so that it serves both to carry the water and to aid in controlling erosion.

Head Walls for Culverts.—The design of head walls for culverts and in fact the whole treatment of head walls has undergone continued evolution over the past 20 years. It was formerly assumed that the culvert should have a stalwart and sizable head wall. Now it is generally conceded that head walls should be dispensed with except when conditions at the inlet end of the culvert require head walls to guide the water into the culvert. In many locations there is no need for head walls, and the barrel of the pipe is simply extended sufficiently to be beyond the toe of the slope of the fill as shown in Fig. 20. The elimination of head walls on culverts in which the barrel consists of sectional pipe is probably inadvisable owing to the probability that without head walls the end sections of pipe might be disturbed and eventually be displaced sufficiently to allow fill material to wash into the culvert at the joint between the first and second section of pipe.

CONTROL OF EROSION

The control of erosion from running water is a problem that confronts highway engineers everywhere, although it is perhaps most acute in the humid areas of the country. Soils of the type that lack cohesion are most susceptible to the effects of flowing water; but if the velocity of flow is great enough, erosion will be a problem to deal with where construction is in any type of soil.

Physical Nature of Erosion.—Erosion of the channels in which water is flowing takes place through the capacity of flowing water to carry a certain amount of finely divided solid matter in suspension and, in addition, to transport heavier particles of solid material by rolling them along the bed of the stream channel. The material to be carried is loosened by the mechanical action of flowing water and by the natural action of the elements including the effect of repeated freezing and thawing, weathering from exposure to the air, and many other familiar agencies that are constantly at work upon the crust of the earth.

The capacity of flowing water to transport solid material increases rather rapidly with the velocity of flow. Quiescent

water will not hold solid matter in suspension indefinitely, although finer particles of solid matter may be held in suspension for considerable periods. From the standpoint of the highway engineering problem it is the capacity of flowing water to transport solids that is of interest.

Basic Methods of Controlling Erosion.—From the nature of the erosive action of flowing water, two basic methods of controlling erosion suggest themselves. The first of these is to provide drainage channels in which the velocity of flow is so low that harmful erosion is eliminated. This method is of limited application because of the necessity in highway drainage of providing for the prompt removal of surface water from the vicinity of the highway. However, it is the most useful and most common method of controlling erosion. The second method of controlling erosion is to carry storm water in drainage channels of erosion resistant materials, a method that has been employed almost universally for the drainage of municipal highways and has come into wide use for rural highways.

Erosion Problems in Highway Construction.—The parts of the highway that are particularly subject to erosion from flowing water are the drainage ditches, the side slopes of cuts and fills, the shoulders adjacent to the surfaced portion of the highway on hills, and banks of streams that cut in on the right-of-way or cross the highway. In addition to these problems that concern themselves with the highway itself, the highway engineer should cooperate as far as possible in those projects for the control of erosion on lands adjacent to the highway. Frequently it will be possible to adjust the highway drainage design to the erosion control program of adjacent lands in such a way as to supplement the effort of the owner without any particular increase in the cost of the highway construction. For all these problems the common method of control of erosion is that of locating drainage channels so as to minimize the flow of storm water over surfaces that because of their slope and texture would erode readily under the influence of flowing water. The drainage channels themselves are either designed to carry the water at low velocity or lined with masonry or concrete so that erosion cannot occur.

Controlling the Erosion of Ditches.—When the side ditches are in natural soil and carry water at a velocity exceeding about 2 ft. per second, erosion will be a factor to be reckoned with.

If the gradient of the ditch does not exceed about 4 or 6 per cent it is customary to control erosion by means of a succession of weirs placed across the ditch at such intervals that the gradient in the ditch proper between weirs will be flat enough to reduce the velocity to a rate that will not produce serious erosion. Immediately below each weir a small amount of erosion will occur, but if the weirs are sufficiently close together the amount of erosion will give no serious maintenance problem. The difference in elevation of the tops of successive weirs should be such that the ditch slope between weirs will not exceed about 2 per cent. These weirs may be constructed of planks, preferably treated with creosote, or of masonry. The weir should be designed with a notch so that the flow will be confined to the middle portion of the weir, and it should extend well back into the soil to prevent water from cutting around the ends.

When the grade of the ditch is in excess of 4 or 6 per cent the cost of the weir will usually be such that it will be more economical to provide a paved gutter for the entire length of the hill. Such gutters must be adjacent to the surfaced portion of the highway, or there will be erosion from the water flowing on the roadway itself. Quite frequently paved gutters are constructed in advance of the surfacing of the roadway between gutters, and when that is done it is the almost universal experience that storm water falling on the roadway itself flows toward the ditch and gutter but gradually cuts for itself a channel at the roadside edge of the paved gutter.

In parkways and on boulevards in hilly country it is not uncommon to find that the roadway surface itself is paved with one of the standard paving materials and the gutter constructed of cobblestones or rough cut blocks, a design that is adopted because of its esthetic quality. Where a space has been left between the paved area and the gutter, there has been difficulty in maintaining this intervening space.

Preventing the erosion of the side slope in cuts and fills has always been a troublesome problem and one that is especially difficult when the soil is of a type that has low cohesion. The standard method of controlling erosion in such cases is to intercept the water that would otherwise flow down the side slope and carry it in a drainage channel that is constructed of materials that will resist erosion or a channel that is provided with weirs to prevent erosion. The control of erosion of side slopes on

TABLE IV.[1]—APPROXIMATE ORDINARY MAXIMUM LOADS ON DRAIN TILE AND SEWER PIPE IN DITCHES FROM COMMON DITCH FILLING MATERIALS. IN POUNDS PER LINEAR FOOT

H = Height of fill above top of pipe	B = Breadth of ditch, at top of pipe									
	1 ft.	2 ft.	3 ft.	4 ft.	5 ft.	1 ft.	2 ft.	3 ft.	4 ft.	5 ft.
	Partly compacted damp top soil. 90 lb. per cubic foot					Saturated top soil. 110 lb. per cubic foot				
2 ft.	130	310	490	670	830	170	380	600	820	1,020
4 ft.	200	530	880	1,230	1,580	260	670	1,090	1,510	1,950
6 ft.	230	690	1,190	1,700	2,230	310	870	1,500	2,140	2,780
8 ft.	250	800	1,430	2,120	2,790	340	1,030	1,830	2,660	3,510
10 ft.	260	880	1,640	2,450	3,290	350	1,150	2,100	3,120	4,150
	Dry sand. 100 lb. per cubic foot					Saturated sand. 120 lb. per cubic foot				
2 ft.	150	340	550	740	930	180	410	650	890	1,110
4 ft.	220	590	970	1,360	1,750	270	710	1,170	1,640	2,100
6 ft.	260	760	1,320	1,890	2,480	310	910	1,590	2,270	2,970
8 ft.	280	890	1,590	2,350	3,100	340	1,070	1,910	2,820	3,720
10 ft.	290	980	1,820	2,720	3,650	350	1,180	2,180	3,260	4,380
12 ft.	300	1,040	2,000	3,050	4,150	360	1,250	2,400	3,650	4,980
14 ft.	300	1,090	2,140	3,320	4,580	360	1,310	2,570	3,990	5,490
16 ft.	300	1,130	2,260	3,550	4,950	360	1,350	2,710	4,260	5,940
18 ft.	300	1,150	2,350	3,740	5,280	360	1,380	2,820	4,490	6,330
20 ft.	300	1,170	2,420	3,920	5,550	360	1,400	2,910	4,700	6,660
22 ft.	300	1,180	2,480	4,060	5,800	360	1,420	2,980	4,880	6,960
24 ft.	300	1,190	2,540	4,180	6,030	360	1,430	3,050	5,010	7,230
26 ft.	300	1,200	2,570	4,290	6,210	360	1,440	3,090	5,150	7,460
28 ft.	300	1,200	2,600	4,370	6,390	360	1,440	3,120	5,240	7,670
30 ft.	300	1,200	2,630	4,450	6,530	360	1,440	3,150	5,340	7,830
Infinity	300	1,210	2,730	4,850	7,580	360	1,450	3,270	5,820	9,090
	Partly compacted damp yellow clay. 100 lb. per cubic foot					Saturated yellow clay. 130 lb. per cubic foot				
2 ft.	160	350	550	750	930	210	470	730	1,000	1,240
4 ft.	250	620	1,010	1,400	1,800	340	840	1,330	1,870	2,370
6 ft.	300	830	1,400	1,990	2,580	430	1,140	1,900	2,630	3,410
8 ft.	330	990	1,720	2,500	3,250	490	1,380	2,360	3,360	4,400
10 ft.	350	1,110	2,000	2,920	3,880	520	1,570	2,760	3,980	5,270
12 ft.	360	1,200	2,220	3,320	4,450	540	1,730	3,100	4,560	6,050
14 ft.	370	1,280	2,410	3,650	4,950	560	1,850	3,410	5,050	6,760
16 ft.	370	1,330	2,570	3,950	5,400	570	1,940	3,660	5,510	7,440
18 ft.	380	1,380	2,710	4,210	5,810	570	2,020	3,880	5,930	8,060
20 ft.	380	1,410	2,830	4,450	6,180	580	2,090	4,070	6,280	8,610
22 ft.	380	1,430	2,920	4,640	6,500	580	2,140	4,240	6,610	9,130
24 ft.	380	1,450	3,000	4,820	6,800	580	2,180	4,380	6,910	9,590
26 ft.	380	1,470	3,060	4,980	7,080	580	2,210	4,500	7,160	10,010
28 ft.	380	1,480	3,120	5,100	7,310	580	2,240	4,610	7,380	10,430
30 ft.	380	1,490	3,170	5,230	7,530	580	2,260	4,700	7,590	10,780
Infinity	380	1,520	3,410	6,060	9,480	580	2,340	5,270	9,360	14,620

[1] Engineering Experiment Station, Ames, Iowa, p. 46, *Bull.* 31

Table V.—Areas of Waterways for Highway Bridges and Culverts

Based on Talbot's formula $a = C\sqrt[4]{A^3}$,

where

a = area of waterway required, in square feet.

A = drainage area, in acres.

C = coefficient used by U.S. Public Roads Administration.

Drainage area, acres	Area of waterways, square feet						
	Mountainous land	Hilly land		Rolling land		Flat land	
	$C = 1.00$	$C = 0.80$	$C = 0.60$	$C = 0.50$	$C = 0.40$	$C = 0.30$	$C = 0.20$
1	1.0	0.8	0.6	0.5	0.4	0.3	0.2
2	1.7	1.4	1.0	0.9	0.7	0.5	0.3
4	2.8	2.2	1.7	1.4	1.1	0.8	0.6
5	3.3	2.6	2.0	1.7	1.3	1.0	0.7
6	3.8	3.0	2.3	1.9	1.5	1.1	0.8
8	4.8	3.8	2.9	2.4	1.9	1.4	1.0
10	5.6	4.5	3.4	2.8	2.2	1.7	1.1
15	7.6	6.1	4.6	3.8	3.0	2.3	1.5
20	9.5	7.6	5.7	4.8	3.8	2.9	1.9
25	11.2	9.0	6.7	5.6	4.5	3.4	2.2
30	12.8	10.2	7.7	6.4	5.1	3.8	2.6
35	14.4	11.5	8.6	7.2	5.8	4.3	2.9
40	15.9	12.7	9.5	8.0	6.4	4.8	3.2
45	17.4	13.9	10.4	8.7	7.0	5.2	3.5
50	18.8	15.0	11.3	9.4	7.5	5.6	3.8
55	20.2	16.2	12.1	10.1	8.1	6.1	4.0
60	21.6	17.3	13.0	10.8	8.6	6.5	4.3
65	22.9	18.3	13.7	11.5	9.2	6.9	4.6
70	24.2	19.4	14.5	12.1	9.7	7.3	4.8
75	25.5	20.4	15.3	12.8	10.2	7.7	5.1
80	26.8	21.4	16.1	13.4	10.7	8.0	5.4
85	28.0	22.4	16.8	14.0	11.2	8.4	5.6
90	29.2	23.4	17.5	14.6	11.7	8.8	5.8
95	30.4	24.3	18.2	15.2	12.2	9.1	6.1
100	31.6	25.3	19.0	15.8	12.6	9.5	6.3
150	42.9	34.3	25.7	21.5	17.2	12.9	8.6
200	53.2	42.6	31.9	26.6	21.3	16.0	10.6
250	62.9	50.3	37.7	31.5	25.2	18.9	12.6
300	72.1	57.7	43.3	36.1	28.8	21.6	14.4
350	80.9	64.7	48.5	40.5	32.4	24.3	16.2
400	89.4	71.5	53.6	44.7	35.8	26.8	17.9
450	97.7	78.2	58.6	48.9	39.1	29.3	19.5
500	105.7	84.6	63.4	52.9	42.3	31.7	21.1
550	113.6	90.9	68.2	56.8	45.4	34.1	22.7
600	121.2	97.0	72.7	60.6	48.5	36.4	24.2

TABLE V.—AREAS OF WATERWAYS FOR HIGHWAY BRIDGES AND CULVERTS —Continued

Drainage area, acres	Area of waterways, square feet						
	Mountainous land	Hilly land		Rolling land		Flat land	
	$C = 1.00$	$C = 0.80$	$C = 0.60$	$C = 0.50$	$C = 0.40$	$C = 0.30$	$C = 0.20$
650	128.7	103.0	77.2	64.4	51.5	38.6	25.7
700	136.1	108.9	81.7	68.1	54.4	40.8	27.2
750	143.3	114.6	86.0	71.7	57.3	43.0	28.7
800	150.4	120.3	90.2	75.2	60.2	45.1	30.1
850	157.4	125.9	94.4	78.7	63.0	47.2	31.5
900	164.3	131.4	98.6	82.2	65.7	49.3	32.9
950	171.1	136.9	102.7	85.6	68.4	51.3	34.2
1,000	177.9	142.3	106.7	89.0	71.2	53.4	35.6
1,050	184.4	147.5	110.6	92.2	73.8	55.3	36.9
1,100	191.0	152.8	114.6	95.5	76.4	57.3	38.2
1,150	197.5	158.0	118.5	98.8	79.0	59.3	39.5
1,200	203.9	163.1	122.3	102.0	81.6	61.2	40.8
1,250	210.2	168.2	126.1	105.1	84.1	63.1	42.0
1,300	216.5	173.2	129.9	108.3	86.6	65.0	43.3
1,350	222.7	178.2	133.6	111.4	89.1	66.8	44.5
1,400	228.9	183.1	137.3	114.5	91.6	68.7	45.8
1,450	235.0	188.0	141.0	117.5	94.0	70.5	47.0
1,500	241.0	192.8	144.6	120.5	96.4	72.3	48.2
1,600	253.0	202.4	151.8	126.5	101.2	75.9	50.6
1,700	264.8	211.8	158.9	132.4	105.9	79.4	53.0
1,800	276.3	221.0	165.8	138.2	110.5	82.9	55.3
1,900	287.8	230.2	172.7	143.9	115.1	86.3	57.6
2,000	299.1	239.3	179.5	149.6	119.6	89.7	59.8
2,100	310.2	248.2	186.1	155.1	124.1	93.1	62.0
2,200	321.2	257.0	192.7	160.6	128.5	96.4	64.2
2,300	332.1	265.7	199.3	166.1	132.8	99.6	66.4
2,400	342.9	274.3	205.7	171.5	137.2	102.9	68.6
2,500	353.6	282.9	212.2	176.8	141.4	106.1	70.7
2,600	364.1	291.3	218.5	182.1	145.6	109.2	72.8
2,700	374.6	299.7	224.8	187.3	149.8	112.4	74.9
2,800	384.9	307.9	230.9	192.5	154.0,	115.5	77.0
2,900	395.2	316.2	237.1	197.6	158.1	118.6	79.0
3,000	405.4	324.3	243.2	202.7	162.2	121.6	81.1
3,250	430.4	344.3	258.2	215.2	172.2	129.1	86.1
3,500	455.0	364.0	273.0	227.5	182.0	136.5	91.0
3,750	479.2	383.4	287.5	239.6	191.7	143.8	95.8
4,000	503.0	402.4	301.8	251.5	201.2	150.9	100.6
4,250	526.4	421.1	315.8	263.2	210.6	157.9	105.5
4,500	549.4	439.5	329.6	274.7	219.8	164.8	109.9
4,750	572.2	457.8	343.3	286.1	228.9	171.7	114.4
5,000	594.6	475.7	356.8	297.3	237.8	178.4	118.9

Table VI.—Areas of Waterways for Highway Bridges and Culverts Based on the Dun Drainage Table[1]

The column for $C = 1$ was prepared from observation of streams in southwestern Missouri, eastern Kansas, western Arkansas, and southwestern Oklahoma, regions in which steep slopes prevail. The other columns are computed from the column for $C = 1$, by applying the coefficients employed for Talbot's formula in Table V.

Drainage area		Area of waterways, square feet						
		Mountainous land	Hilly land		Rolling land		Flat land	
Square miles	Acres	$C = 1.00$	$C = 0.80$	$C = 0.60$	$C = 0.50$	$C = 0.40$	$C = 0.30$	$C = 0.20$
8	5,120	601	481	361	301	240	180	120
8.5	5,440	622	498	373	311	249	187	124
9	5,760	641	513	385	321	256	192	128
9.5	6,080	660	528	396	330	264	198	132
10	6,400	679	543	407	340	272	204	136
11	7,040	710	568	426	355	284	213	142
12	7,680	740	592	444	370	296	222	148
13	8,320	775	620	465	388	310	233	155
14	8,960	805	644	483	403	322	242	161
15	9,600	835	668	501	418	334	251	167
16	10,240	865	692	519	433	346	260	173
17	10,880	890	712	534	445	356	267	178
18	11,520	920	736	552	460	368	276	184
19	12,160	945	756	567	473	378	284	189
20	12,800	970	776	582	485	388	291	194
22	14,080	1,015	812	609	508	406	305	203
24	15,360	1,060	848	636	530	424	318	212
26	16,640	1,100	880	660	550	440	330	220
28	17,920	1,140	912	684	570	456	342	228
30	19,200	1,180	944	708	590	472	354	236
32	20,480	1,220	976	732	610	488	366	244
34	21,760	1,255	1,004	753	628	502	377	251
36	23,040	1,290	1,032	774	645	516	387	258
38	24,320	1,320	1,056	792	660	528	396	264
40	25,600	1,350	1,080	810	675	540	405	270
45	28,800	1,435	1,148	861	718	574	431	287
50	32,000	1,510	1,208	906	755	604	453	302
55	35,200	1,580	1,264	948	790	632	474	316
60	38,400	1,650	1,320	990	825	660	495	330
65	41,600	1,720	1,376	1,032	860	688	516	344

[1] *Proc. Amer. Ry. Eng. Maintenance of Way Assoc.*, Vol. 12, Part 3, p. 484, 1911.

TABLE VI.—AREAS OF WATERWAYS FOR HIGHWAY BRIDGES AND CULVERTS —Continued

Drainage area		Area of waterways, square feet						
		Mountainous land	Hilly land		Rolling land		Flat land	
Square miles	Acres	$C = 1.00$	$C = 0.80$	$C = 0.60$	$C = 0.50$	$C = 0.40$	$C = 0.30$	$C = 0.20$
70	44,800	1,780	1,424	1,068	890	712	534	356
75	48,000	1,840	1,472	1,104	920	736	552	368
80	51,200	1,900	1,520	1,140	950	760	570	380
85	54,400	1,960	1,568	1,176	980	784	588	392
90	57,600	2,015	1,612	1,209	1,008	806	605	403
95	60,800	2,065	1,652	1,239	1,033	826	620	413
100	64,000	2,120	1,696	1,272	1,060	848	636	424
110	70,400	2,220	1,776	1,332	1,110	888	666	444
120	76,800	2,315	1,852	1,389	1,158	926	695	463
130	83,200	2,405	1,924	1,443	1,203	962	722	481
140	89,600	2,500	2,000	1,500	1,250	1,000	750	500
150	96,000	2,580	2,064	1,548	1,290	1,032	774	516
160	102,400	2,665	2,132	1,599	1,333	1,066	800	533
170	108,800	2,745	2,196	1,647	1,373	1,098	824	549
180	115,200	2,820	2,256	1,692	1,410	1,128	846	564
190	121,600	2,900	2,320	1,740	1,450	1,160	870	580
200	128,000	2,970	2,376	1,782	1,485	1,188	891	594
220	140,800	3,115	2,492	1,869	1,558	1,246	935	623
240	153,600	3,245	2,596	1,947	1,623	1,298	974	649
260	166,400	3,370	2,696	2,022	1,685	1,348	1,011	674
280	179,200	3,495	2,796	2,097	1,748	1,398	1,049	699
300	192,000	3,615	2,892	2,169	1,808	1,446	1,085	723
325	208,000	3,770	3,016	2,262	1,885	1,508	1,131	754
350	224,000	3,900	3,120	2,340	1,950	1,560	1,170	780
375	240,000	4,035	3,228	2,421	2,018	1,614	1,211	807
400	256,000	4,165	3,332	2,499	2,083	1,666	1,250	833
450	288,000	4,385	3,508	2,631	2,193	1,754	1,316	877
500	320,000	4,610	3,688	2,766	2,305	1,844	1,383	922
550	352,000	4,825	3,860	2,895	2,413	1,930	1,448	965
600	384,000	5,030	4,024	3,018	2,515	2,012	1,509	1,006
700	448,000	5,420	4,336	3,252	2,710	2,168	1,626	1,084
800	512,000	5,800	4,640	3,480	2,900	2,320	1,740	1,160
900	576,000	6,080	4,864	3,648	3,040	2,432	1,824	1,216
1,000	640,000	6,380	5,104	3,828	3,190	2,552	1,914	1,276
2,000	1,280,000	8,820	7,056	5,292	4,410	3,528	2,646	1,764
3,000	1,920,000	10,640	8,512	6,384	5,320	4,256	3,192	2,128
4,000	2,560,000	12,160	9,728	7,296	6,080	4,864	3,648	2,432
5,000	3,200,000	13,500	10,800	8,100	6,750	5,400	4,050	2,700

fills has been discussed in connection with the drainage of embankments. Here the problem is also one of intercepting the water that would otherwise flow down the side of the fill.

Wherever the soil and rainfall characteristics of a region are favorable to the growth of the various rough grasses, every effort should be made to secure a covering of grass on side slopes and in the ditches. As soon as the road has been brought to final grade, and construction of the wearing surface completed, the shoulders, ditches, and back slopes are seeded with a view to developing a covering of grass as quickly as possible. Except under very favorable rainfall conditions, there are few grasses that will prove hardy enough to live on the side slope in cuts and fills, but western wheat grass and Hungarian brome will survive in most of the humid areas. The various state highway departments have developed a special technique suitable to their particular climatic condition for developing a growth of grass wherever it is possible to do so.

Control of Stream Erosion.—To a limited extent the drainage structures on a highway can be utilized to minimize stream erosion and thus aid in the control of erosion on the adjacent land. In some instances the drainage area for a culvert is so small that the drainage may be cared for by means of underground tile instead of an open culvert being used, and this of course contributes to the control of erosion. Where the lands adjacent to the highway are being protected from erosion through a system of terraces, it is often convenient to provide an outlet from the terraces into the road ditch, and wherever the ditch can be utilized in this way it is an economy to the adjacent land owners and a type of cooperation that should be extended.

The effectiveness of the drop inlet type of culvert in controlling erosion was discussed in connection with the design of culverts. Its function is to produce a reservoir of quiescent water above the culvert in which sedimentation will take place and thus preserve soil that otherwise would be carried away by the stream. Its use is advisable in many instances where in the past the ordinary type of culvert has been employed.

CHAPTER IV

SOIL AS A HIGHWAY MATERIAL

The science of soils mechanics has progressed to a stage that permits certain generalizations that have a definite significance to the highway engineer. The following discussion will include a presentation of the elementary characteristics of soil[1] and of the application of the present knowledge about soils to their classification for the purposes of the highway engineer.

THE NATURE AND PROPERTIES OF SOIL

Formation of Soil.—Soil is a product of the decomposition of the earth's crust by the action of the elements and therefore consists of particles of rock ranging in size down to an impalpable powder. The detritus thus produced has for untold centuries been shifting about through the action of water and wind and the force of gravity. Erosion and transportation by water and wind have played their part in laying down the soil upon which the engineer must place his structures. Great reaches of soil of uniform composition will be found in some parts of the world, and in others there will be many varieties of soil on a single highway project. It is to be expected that in the humid areas numerous kinds of soil will be encountered in a single state and that the comparatively shallow excavations required in highway construction will uncover a good many types of soil formations that do not afford a stable road foundation. The engineer has learned to recognize these and to develop a treatment for the stabilization of each or, if that is not feasible, to design a road surface that will have the requisite stability on a poor subgrade.

The mineral composition of soil is important to the highway engineer only to the extent that it indicates certain definite and well-understood physical properties.

Influence of Water on Soils.—What must be recognized at the beginning of the study of soils is that roads are built on a layer

[1] For a more detailed discussion of this subject see C. A. Hogentogler and C. A. Hogentogler, Jr., "Engineering Properties of Soils," McGraw-Hill Book Company, Inc., New York, 1937.

of soil *that may subsequently become moistened to varying degrees* depending upon the hydrological and geological conditions peculiar to the region and upon the design of the highway. Therefore the most significant soil tests are those which show the behavior of a specific soil when it carries various percentages of water. If peat and other materials that are largely organic in character are excluded, soils of all sorts when "dry" have high load-carrying capacity. There is some small moisture content (called the *optimum* moisture content) at which most soils are more stable than when truly dry (soils in humid regions rarely become truly dry). Therefore it is sufficiently accurate to say that dry soils have all of the load-carrying capacity required for highway loads. But in many regions the precipitation is such that the soil foundation under a road will absorb considerably more than the optimum amount of water unless definite precautions are taken to prevent it. The important question for the highway engineer then becomes one of the load-carrying capacity of the soil when it contains more than the optimum amount of water. It is to the determination of the optimum water content and the effect of greater quantities than the optimum that many soil tests are directed.

As far as the road builder is concerned, the most significant differences in soils are the variations in the proportions of the several sizes of grains and the behavior of the soils when they carry varying percentages of moisture.

CONSTITUENTS OF SOIL

There is no wholly consistent terminology employed in the literature of soils mechanics, but it is necessary to use certain terms in this discussion. These will be listed and defined in the following sections so that confusion of meaning may be avoided.

Colloids.—Colloids are particles of mineral matter smaller than 0.001 mm. in size and comprise the most finely divided particles of soil. The nature of colloids is not too well understood, but it is known that the colloids from various soils, although perhaps of about the same grain size, may exhibit widely different physical characteristics. So far as the highway engineer is concerned there is not at present any basis for the classification of colloids into groups or any definite reason for thinking that such a classification is necessary.

Clay as a Soil Constituent.—Clay is soil consisting of particles lying between 0.005 and 0.001 mm. in size and therefore next larger than colloids. This definition does not carry any restriction as to the color or mineral composition of the particles, being based wholly on grain size.

Silt as a Soil Constituent.—Silt consists of soil particles next coarser than clay, the particles being smaller than 0.05 and coarser than 0.005 mm. This definition includes no stipulation as to the mineral composition of the particles but, on the contrary, is based wholly on size of particle; but in general silt may be described as exceedingly fine "sand."

Sand as a Soil Constituent.—Sand as a soil constituent is material coarser than silt, the grain size ranging from 0.05 to 0.2 mm. in size. The definition does not stipulate the mineral composition of the sand, but in general it is the familiar granular material consisting of fragments of silicious rocks.

TYPES OF SOILS

Sand as a Soil Type.—Sand as a soil type is true sand (particles of silicious rocks) with which there may be mixed a small amount (not more than 20 per cent) of silt (which is really exceedingly fine sand) and clay. Sand is cohesionless, permeable, and high in internal friction. It does not retain capillary moisture and is self-draining. It may be classed as coarse, medium, fine, or very fine, according to the grain size. In soils analysis, sand is arbitrarily defined as the fraction lying between 2.00 and 0.005 mm. in size. This is in contrast with concrete or masonry sand, both of which are much coarser than soil sand, but these coarser sands may be an ingredient of soil, and if so the particles larger than 2.00 mm. in size would be called "gravel" in the soils nomenclature.

Sandy Loam Soil.—Sandy loam is a mixture containing 20 to 50 per cent of sand and varying amounts of silt and clay. It possesses some cohesion, considerable internal friction, and high permeability. It is readily drained by means of tile and does not retain capillary moisture very tenaciously. Like sand, it may be classed as coarse, medium, fine, or very fine, according to the grain size.

Loam Soil.—Loam consists of about equal parts of sand, silt, and clay and not more than 1 per cent of colloids. It is somewhat cohesive and possesses moderate internal friction.

It is permeable and responds readily to tile drainage. It does not retain capillary moisture for any protracted period.

Silt Loam Soil.—Silt loam consists of some sand, silt in excess of 50 per cent, up to 10 per cent of clay, and not more than 3 per cent of colloidal material. It is somewhat cohesive, low in internal friction, and usually sufficiently permeable to be drained by tile. It tends to retain capillary moisture.

Clay Loam Soil.—Clay loam consists of fine and very fine sand, silt, clay in excess of 20 per cent, and not more than 5 per cent of colloids. It is rather cohesive and is low in internal friction and permeability. This soil does not respond to tile drainage except in special cases and tends to hold capillary moisture quite tenaciously.

Clay Soil.—Clay consists of silt, a high percentage of clay (50 or more), and varying percentages of colloids. It is cohesive, plastic, impermeable, and low in internal friction. It does not respond to tile drainage and retains capillary moisture with great tenacity.

Miscellaneous Terms.—The term "peat" is used to designate decomposed and partially decomposed vegetable matter that is not really soil. "Sandy loam," "silty clay," and "gravelly clay" are terms that are sometimes convenient, and their meaning is probably clear from the foregoing definitions.

Definitions Used in Sieve Analysis.—In reporting the results of the mechanical analysis of soils it is the practice to separate the sample into fractions corresponding to the foregoing definitions. The terms employed in soils classification therefore specify a grouping of the particles by size. The several fractions determined in the mechanical analysis of soils are as follows:

1. Particles larger than 2.0 mm. ("gravel") 2. *Coarse sand,* 2.0 to 0.25 mm. 3. *Fine sand,* 0.25 to 0.05 mm.	Determined by sieve analysis
4. *Silt,* 0.05 to 0.005 mm. 5. *Clay,* smaller than 0.005 mm. 6. *Colloids,* smaller than 0.001 mm.	Determined by settling velocity according to Stokes' law

CHARACTERISTICS OF SOILS

Structure of Soils.—The individual particles of soil may be smoothly rounded, angular, or spike- or platelike. In the undisturbed natural state these particles may have been aggre-

gated into grains or lumps of various sizes and of all possible degrees of stability. The particles in loam seem to be loosely bound into grains, and hence it is said that loam is not cohesive. The converse is true of the clays, which are highly cohesive. Sand grains are not bound into lumps. The soil mass is interspersed with "void" space, which is filled with air or, at certain times, with water. Roots, burrowing insects and worms, and

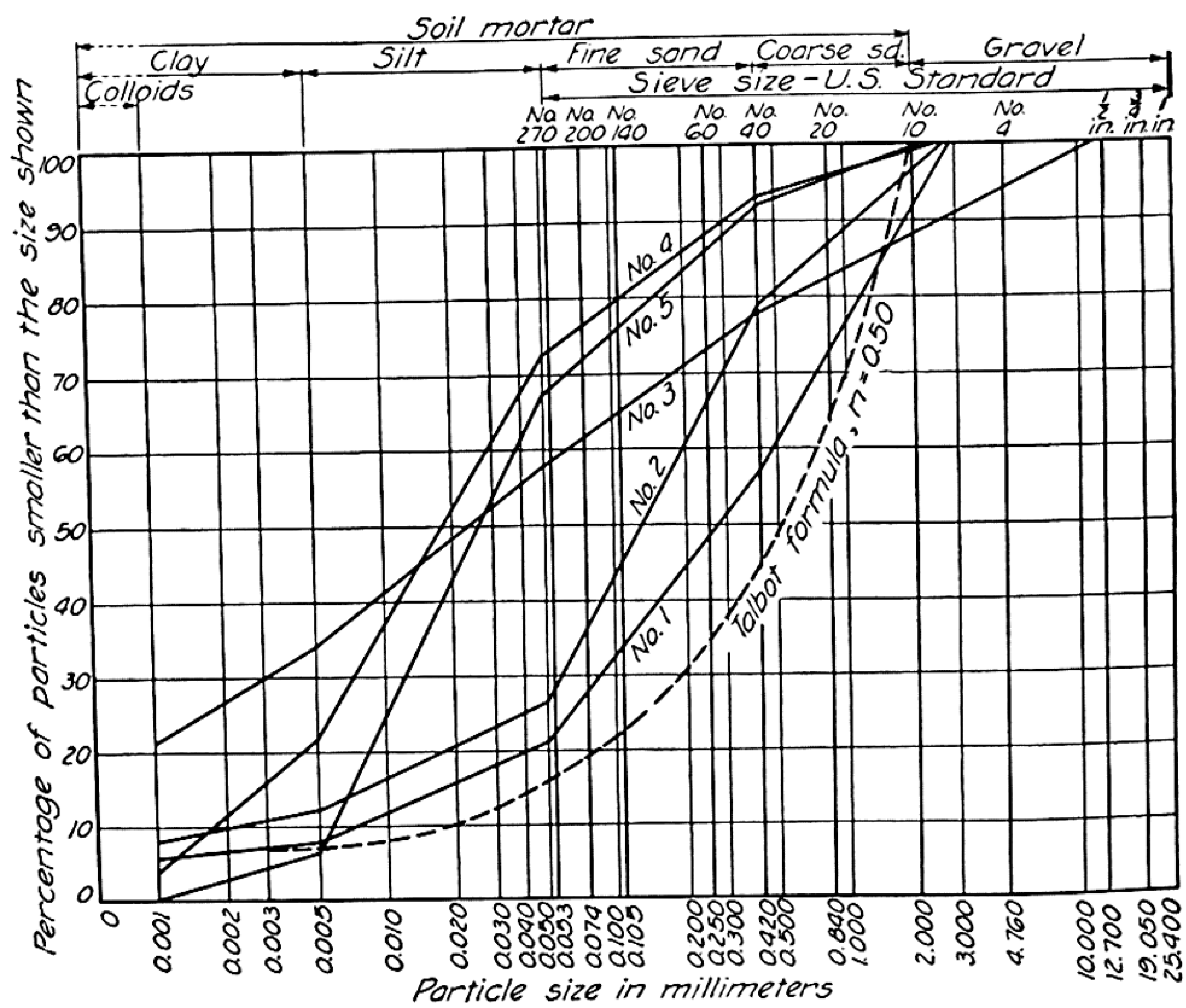

FIG. 21.—Particle-size distribution curves for soils.

percolating water each contributes to the formation of void space, but fundamentally the void space is due to the impossibility of packing the irregular soil particles sufficiently closely together to eliminate void space. The extent of the void space in a soil is determined by the *voids ratio* and *porosity* tests.

Soil may be looked upon as a mixture of varying percentages of the several sizes of grains listed (page 85) above. The gravel alone would be a porous mass in which the air voids were large enough to be readily visible. In typical soils these voids are partly or completely (perhaps overfilled) with sand, thus reducing the size of the voids in the mass. The voids in the sand will

accommodate the grains of silt; the voids in the silt will accommodate the clay; and the voids in the clay, the colloids. A soil in which the proportion of each size of grain is just large enough to fill the voids not filled by the next coarser size is said to be *well graded*. Well-graded (page 164) soils are more stable in embankments and subgrades than poorly graded soils, for the simple reasons that there is more solid matter per unit of volume and less void space to hold water.

The distribution of the several sizes of particles in a soil sample is most conveniently shown by the mechanical analysis curve or, more properly, the *particle-size distribution curve*. Several of these curves are shown in Fig. 21. It is well to note that they are plotted on sheets in which the abscissas are a repeated logarithmic scale and the ordinates are arithmetical (called *five-cycle semilogarithmic coordinate sheets*). When plotted to this scale, the curves of well-graded soils are concave upward, and the more nearly they approach the curve for an ideal mixture (shown also in Fig. 21) the better the grading, the more stable the soil mechanically, and the less likely it is to become unstable when wet.

Maximum Density Grading.—The grading of a soil may be tested by comparing the grading determined from the mechanical analysis with the curve obtained by using the Talbot formula for maximum density of concrete aggregates, which is

$$p = \left(\frac{d}{D}\right)^n, \tag{1}$$

in which

p = the portion by weight passing a sieve opening.
d = the size (linear) of screen opening.
D = the maximum particle size.
n = an empirical exponent.

For soils comparisons n is varied between 0.3 and 0.5, to permit developing a series of curves in which the variables are n and D.

Permeability.—When soil is porous, water will percolate through it unless the pores are so very small that the water is held in the soil by capillarity and the rate of percolation is so small as to warrant considering the soil as impermeable. The degree of permeability of a soil, which indicates the facility with which water will percolate through it, is of significance in the design of highway drainage. Some soils can be drained by

tile, and some cannot. The coefficient of permeability can be determined by laboratory methods for soil in various degrees of compaction and is useful in designing drainage. This factor is also very useful in designing the mixtures of soil and aggregate used for soil-bound roadway surfaces, such as gravel roads, since low permeability is desired for wearing surfaces, and high permeability for subgrades.

The *coefficient of permeability* k is the discharge through the soil in cubic feet per 24 hr. for a hydraulic gradient of unity[1] (equals pressure head on the specimen in feet divided by the depth of specimen in feet). For porous soils that may be drained by tile, k will range from 25 to 50. Sandy clays usually have coefficients of less than 5, and these do not readily respond to tile drainage. Highly cohesive clays have a coefficient of less than 1, and some samples are impervious; that is, $k = 0$. It is probably hopeless to attempt to reduce the water content of soils by tile drains if the permeability coefficient is less than 15. (But note that wet seams in such soils can be intercepted by tile drains to good advantage.)

Capillarity in Soil.—An understanding of the nature of capillary moisture and its effect upon the soil mass that encloses it may be obtained by considering first an exceedingly simple and elementary case. Conceive a lump of soil of small size through which extends a single tiny channel such as might have been formed by a rootlet that had later decayed. This channel may lie vertically or horizontally or at any angle. If at one end of this channel there is a supply of water, the entire channel will fill through the special manifestation of surface tension known as *capillary action.* If the supply of water is then removed from the end of the channel, that which has been drawn into the channel through capillary action will remain there but will begin to evaporate at the exposed ends. At each end of this column of water will be the familiar meniscus, the perimeter of which is in contact with the soil mass. The tension of these menisci tends to shorten the tube of soil carrying the little column of water and results in longitudinal compression of the soil forming the capillary tube. The magnitude of this compression is equal to the maximum capillary tension at the meniscus, which is:

[1] GRIFFITH, J. H., "Physical Properties of Earths," *Bull.* 101, *Iowa Engr. Exp. Sta.*, pp. 114–120, 1931.

Max. capillary tension $= \frac{0.306}{a}$ g. per square centimeter of cross-section area of the capillary tube,

where a is that area.[1]

We have, then, the conception of a minute column of water at each end of which there is a meniscus in tension, which in turn results in compression in the mass of soil forming the capillary tube. The soil surrounding this minute filament of moisture may be thought of as a tube that is being shortened by the surface tension phenomenon, with resulting compression in the soil.

If one can imagine a lump of soil interlaced with innumerable capillary channels of the character just described, and a source of supply of water, each of these capillary tubes will be filled with water. If the supply of water is removed, the water in each capillary tube will begin to evaporate, exerting longitudinal pressure in the walls of the little tube of earth surrounding it, and the whole lump of soil will be subjected to compressive forces. As the water gradually evaporates, the soil will be compressed by this capillary tension, and shrinkage will occur if the soil particles are not already in contact. If the soil particles are already in close contact, there will be no change in the physical conformation of the lump of soil; nevertheless the pressure condition will exist within the mass.

It is of importance to recognize the fact that in accordance with the foregoing formula, capillary pressure increases as the diameter of the tube decreases, other factors being held constant, and that the height to which a column of moisture can rise in a soil will depend upon the size of the pore space that makes up the capillary tube in the soil. The capillary tension also varies with the temperature of the water and with the coefficient of friction of the walls of the tube in which the water is held.

The foregoing discussion is based upon the assumption that tubes exist that are definitely continuous throughout the mass of soil and that their conformation is sufficiently regular for them to be recognized as definite individual tubes. As a matter of fact it is only incidentally that such tubes exist. In general the structure of the soil is such that the capillary channels are irregular and formless and are so variable in occurrence as to defy definite analysis of the cross-sectional area or size. Never-

[1] TERZAGHI, DR. CHARLES, "Principles of Final Soil Classification," *Public Roads*, Vol. 8, No. 3, p. 52, May, 1927.

theless that they do exist and that capillary tension may be present are clearly established. The important thing for the engineer to recognize is that water will be transported through the soil in apparent defiance of the laws of gravity when the pores are of a capillary character. It is also important to recognize that there must be available somewhere in the soil a supply of free water for the capillary tubes before such water can rise in the soil.[1] There are three ways in which capillary water can be eliminated from the soil mass:

1. Eliminate the supply of water for the capillary tubes.
2. Completely fill the soil pores so there can be no capillary channels (this is possible only by complete vitrification).
3. Make the pores so large that capillary action is negligible.

Hygroscopic Water in Soil.—When the free water is driven out of a soil by low heat there will remain a film held on each grain by the surface tension of the water. This film may be thought of as consisting of many layers of water of exceeding thinness. The outer layer will perhaps evaporate in much the same way as free water, but each succeeding layer will be more difficult to evaporate, and in general this film finally reaches a critical thinness beyond which it will not evaporate except at temperatures much above the boiling point and then but slowly. Likewise this film of water will resist freezing until the temperature is considerably below 32°F. These facts must be taken into account in two phases of highway drainage:

1. In a humid climate the lowest water content that can be achieved by drainage methods in a loam soil with coarse pores is that in which the moisture has been reduced to the hygroscopic moisture; in a soil with minute pores, to the capillary moisture.
2. Provision against "frost heaving," or the formation of ice lenses in the soil, can be achieved with certainty only after a full understanding of the behavior of capillary and hygroscopic moisture.

Of course neither of these problems will be encountered in handling soils in arid regions (annual precipitation 15 in. or less), and the formation of ice is not a problem in regions where the ground does not freeze to a depth of at least a foot.

Free Water in Soil.—Free water flows through the pores of the soil in accordance with the laws of hydraulics when the pores

[1] Griffith and others have measured capillary rises of 2½ to 3 ft. in the laboratory, and field studies have indicated that capillary rise of water may reach 8 or 10 ft.

are too large for the water to be held by capillarity. Generally such water is in motion, although at times of heavy precipitation there may be short intervals when the movement of the water in the soil is very slight. Ordinarily, free water does not remain either on the surface of a highway or in the pores of the soil in any one location for any considerable period of time. Free water flows in seams and porous layers of the soil (page 52) that lie above impermeable layers, often in a sufficient quantity to produce at some outcropping of this seam a spring of considerable flow. Many of these natural springs pass through wet and dry seasons without any great change in the rate of flow; they appear to be fed from an underground water supply that is so enormous and so distributed as to be little affected by seasonal and annual variations in precipitation.

It is free water that occasions the necessity for tile drainage, and it is free water that supplies the water which is drawn into the very small pores of soils by capillary action. Free water can be removed from permeable soils by underdrainage; capillary water cannot be removed by such drainage. The pores in sand and other coarse-grained soils are so large that water flows freely through them. Nevertheless the surfaces of the sand grains will remain moist for a considerable period of time after the free water is gone because the moisture is held on the surface of the grains by surface tension. That is why moist sand is more stable than dry sand.

SOILS MECHANICS TECHNOLOGY

The technology of soils mechanics has been developing slowly during the past decade, and marked progress has been made in recent years in the struggle to devise appropriate tests and correlate them with soil behavior. Only the most elementary presentation of this subject can be presented within the scope of this discussion.

Soils Constants.—The study of certain phases of soil stability, particularly those relating to the lower layers of embankments and side slopes and to loads on underground structures such as tile lines and culverts and on retaining walls, involves the use of certain soil constants or coefficients that are determined for each class and condition of soil to be dealt with. The more commonly used of these are the angle of internal friction, the coefficient of permeability (page 87), and Poisson's ratio.

Poisson's ratio of lateral to longitudinal strain μ (the symbol σ is also used) is employed in certain mathematical analyses. For soils, μ ranges in magnitude from 0.15 to 0.55 and should be determined for specific soils for which it is to be used in analytical work. It is determined by measuring in the laboratory the deformation in the vertical and horizontal directions of samples of soil subjected to compression.

Angle of Internal Friction.—Internal friction is the property that enables a soil mass to resist deformation through the stabilizing influence of its interlocked angular particles. Dry sand has internal friction but no cohesion. Wet sand has internal friction and some cohesion. Clays have cohesion but little internal friction. It is difficult, and generally unnecessary, to determine separately the effect of cohesion and internal friction on the shear strength of soils. Internal friction may be determined on samples of loose soil, which gives the smallest value for shear strength for that soil which is likely to obtain in new fills or on the side slopes of cuts. Some authorities question the possibility of differentiating between internal friction and cohesion, and fortunately it is seldom necessary to do so.

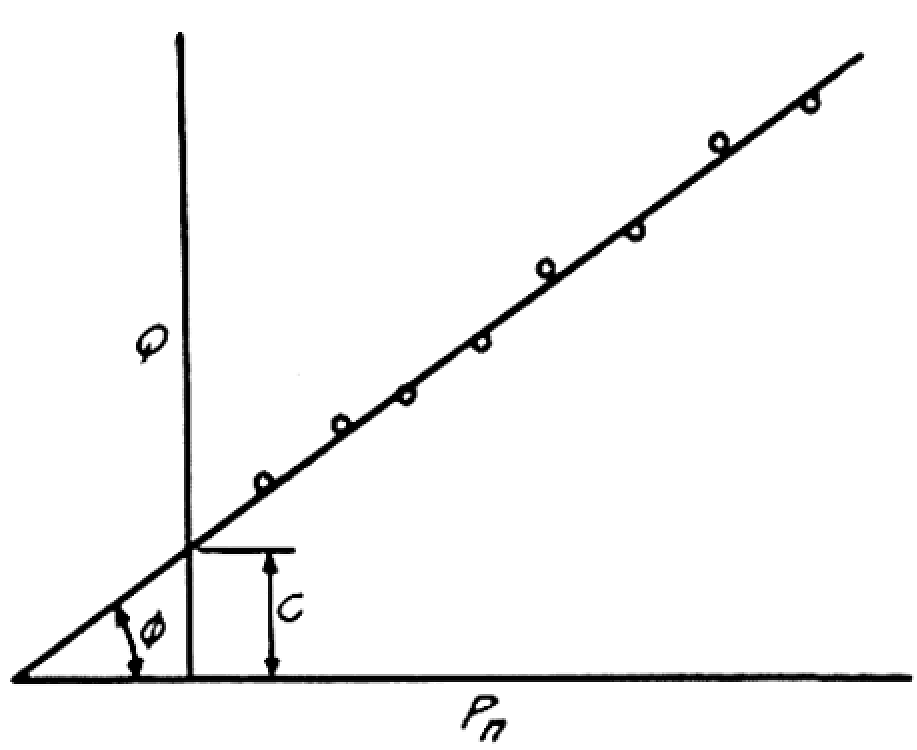

FIG. 22.—Diagram for determining the angle of internal friction in soils.

The approximate value of the angle of internal friction ϕ, may be determined as follows:

By definition,

$$Q = P_n \tan \phi + c, \tag{2}$$

where

Q = the unit shearing force for the soil.
P_n = the unit pressure normal to the shear surface.
ϕ = the angle of internal friction in the soil.
c = the unit cohesion between the shearing surfaces (analogous but not equal to the tensile strength of the soil).

To determine ϕ, Q can be determined experimentally for a number of values of P_n, and the results platted as shown in Fig. 22. If needed, an average value for c can also be read from the diagram. $c = 0$ for cohesionless sand, and $\tan \phi$ ranges

in value between 0.25 and 0.75, with 0.55 a fair average value. Few determinations of values for c are available for cohesive or partly cohesive soils, but tan ϕ apparently varies all the way from about 0.3 to 1.00. ϕ is known to vary with the type of soil and the moisture content thereof. For any specific project involving these constants, determinations of ϕ and c are necessary if a safe design is to be worked out.

Cohesion.—Cohesion is a condition in which the particles of soil are noticeably bound together by molecular force acting at points of contact, aided by colloidal glue, by hygroscopic moisture, or by a combination of these factors. Cohesion is indicated by the effort required to work the soil, by the lumpiness of the material loosened by excavating machinery, and by the length of time a lump of the material will retain its form when immersed. Soils with *high cohesion* often have relatively high *compressive, tensile*, and *shear* strength and if unconfined do not readily displace laterally under load. The manipulation of a wet, cohesive soil by rollers increases the cohesion by pressing the particles into close contact.

Compressibility.—A soil mass may be compressed by exerting sufficient pressure to reduce the void space between the grains, thus forcing out part of the air and also the water if there is any. The extent to which the volume of the soil mass may be reduced (compressed without lateral expansion) affords a basis for determining the shrinkage (or swell) that takes place when soil is transferred from cut to fill. It also indicates roughly the voids ratio that may be reached by compacting the soil in embankments or on subgrades.

Elasticity.—The stress-strain curves for soils in compression, tension, and shear are generally straight lines up to some critical unit stress as is the case with steel, but generally the specimen does not recover fully when the strain is removed. On the contrary there is some permanent deformation due to a rearrangement of the particles during the test. The test data now available seem to show that even under repeated small loadings a soil will be deformed and that the granular, non-cohesive soils or the cohesive soils with a high plasticity index are the most stable of all the soils under repeated loading. If traffic is carried on the thin, and consequently flexible, low-cost types of wearing surface, it is important that the road surface and the supporting layer of soil be designed of a mixture that will not deform permanently

under the traffic loads. This usually requires careful appraisal of the elasticity of the soils encountered at the location.

Effective Size.—The effective size of a soil is the maximum size of the particle of the finest 10 per cent by weight. In Fig. 21 the particle-size distribution curve is shown for a soil marked No. 1. It will be seen by inspection of this diagram that the effective size for this soil is 0.010 mm. This term is more commonly employed in connection with filter sands than in soils engineering, but the effective size does have significance in soils work because the smaller the effective size the more fine grained and close textured the soil. Low effective sizes generally indicate high capillarity and poor drainage properties. For example, the effective size of soil No. 3, Fig. 21, is in the colloidal range; and of No. 2, in the clay range; whereas No. 1, referred to above, which is a loam soil, has its effective size in the silt range. Soils whose effective size is below the silt range are poor construction material, as a rule, and need admixture with coarse-grained material to provide stable subgrades or fills.

Uniformity Coefficient.—The uniformity coefficient of a soil is the maximum particle size of the smallest 60 per cent by weight, divided by the effective size. For soil No. 1, the maximum particle size of the smallest 60 per cent is 0.48 mm., and the effective size was determined to be 0.010. The uniformity coefficient is therefore $0.48/0.01 = 48$. This indicates a wide range of particle size in the 50 per cent of the sand considered and in conjunction with the particle-size distribution curve leads to the conclusion that the soil is reasonably well graded from coarse to fine. A uniformity coefficient of unity would indicate that all the particles were of the same size, and the uniformity coefficient for a soil complying with the Talbot curve for sand with $n = 0.50$ is about 35. Consideration of both the uniformity coefficient and the effective size is sometimes helpful in gaining a conception of the characteristics of a soil.

Routine and Special Soils Tests.—A system of procedure for the routine determination of the engineering properties of soils is being evolved gradually. Many of the tests are wholly empirical, and none has as yet been acceptably standardized but can be used with considerable assurance by an experienced laboratory technician. The more important of these soils tests are described briefly in the following sections, but the detailed

statement of the test procedure is too voluminous to be included herein but may be found in the publications referred to.[1]

Standard Strength Tests.—The data giving the strengths of soils in tension, compression, and shear and the modulus of elasticity[2] are useful in evaluating the soil in its undisturbed natural state, but probably such tests also have some value in establishing the properties of types of soil. These are not routine or field tests but, on the contrary, must be made in a laboratory and under careful control. The soils should be tested in their natural state, the samples having been taken carefully so that their natural texture and moisture content are retained. Other samples should be made up by manipulating the soil and adding various percentages of water. Measurements of vertical and lateral deformation should be obtained, and some tests should be made on samples restrained against lateral deformation.

Field tests of the supporting strength of soils have some value but must be made on fairly large areas (5 sq. ft. or more) and even then cannot be given too much weight. The Proctor needle (page 99) is very useful in studying the density of a road surface or an embankment. These various tests aid the engineer in gaining an understanding of the behavior of the subgrade soil

[1] Detailed instructions for certain tests of soils will be found in the following publications of the A.S.T.M.

"Centrifuge Moisture Equivalent of Soils," *Proc. A.S.T.M.*, Vol. 35, Part I, p. 973, 1935. Also A.S.T.M. Tentative Standards, 1935, p. 908.

"Field Moisture Equivalent of Soils," *Proc. A.S.T.M.*, Vol. 35, Part I, p. 976. Also A.S.T.M. Tentative Standards, p. 911, 1935.

"Liquid Limit of Soils," *Proc. A.S.T.M.*, Vol. 35, Part I, p. 966, 1935. Also A.S.T.M. Tentative Standards, p. 901, 1935.

"Mechanical Analysis of Soils," *Proc. A.S.T.M.*, Vol. 35, Part I, p. 953, 1935. Also A.S.T.M. Tentative Standards, p. 889, 1935.

Moisture Equivalent of Subgrade Soils in the Field, "A.S.T.M. Book of Standards," Part II, p. 970, 1936.

"Plastic Limit and Plasticity Index," *Proc. A.S.T.M.*, Part I, p. 970, 1935. Also A.S.T.M. Tentative Standards, p. 905, 1935.

"Preparing Soil Samples for Analysis," *Proc. A.S.T.M.*, Part I, p. 950, 1935. Also A.S.T.M. Tentative Standards, p. 886, 1935.

"Shrinkage Factors of Soils," *Proc. A.S.T.M.*, Part I, p. 978, 1935. Also A.S.T.M. Tentative Standards, p. 913, 1935.

"Surveying and Mapping Soils," *Proc. A.S.T.M.*, Part I, p. 940, 1935. Also A.S.T.M. Tentative Standards, p. 876, 1935.

[2] Methods of determining the physical properties of soils are described in *Bull.* 101, *Iowa Eng. Exp. Sta.*, June 3, 1931.

under load but cannot be interpreted directly in terms of any standard procedure for improving the soil.

The shear test on subgrade soils and on the soils upon which heavy fills are to be placed is especially valuable if the determinations are made on the soil in a condition approximating closely those of service or the critical conditions of service.[1]

Routine Tests of Soils.—Certain routine tests are employed for determining the properties of soils that have been manipulated to destroy the natural structure. The most important of these are the following:

1. *Liquid Limit.*—The liquid limit of a soil is that moisture content, expressed as a percentage of the weight of the oven-dried soil, at which the soil will just begin to flow when lightly jarred ten times, using a dish of special form. The test shows the amount of moisture necessary to fill the voids in the soil and thus lubricate the particles just enough so that flow is imminent.
2. *Plastic Limit.*—The plastic limit of a soil is the lowest moisture content, expressed as a percentage of the weight of the oven-dried soil, at which the soil can be rolled into threads ⅛ in. in diameter without their breaking into pieces.
3. *Plasticity Index.*—The plasticity index of a soil is the difference between its liquid limit and its plastic limit; that is, plasticity index = liquid limit − plastic limit. This index shows the range of moisture content throughout which the soils remain plastic.
4. *Centrifuge Moisture Equivalent.*—The centrifuge moisture equivalent of a soil is the amount of moisture, expressed as a percentage of the weight of the oven-dried soil, retained by a soil that has been first saturated with water and then subjected to a force equal to one thousand times the force of gravity for one hour. The test shows the extent to which the compressed soil is pervious to water and its capacity to hold capillary moisture, which will not be expelled by the test.
5. *Field Moisture Equivalent.*—The field moisture equivalent of a soil is defined as the minimum moisture content, expressed as a percentage of the weight of the oven-dried soil, at which a drop of water placed on a smoothed surface of the soil will not immediately be absorbed by the soil but will spread out over the surface and give it a shiny appearance. The field moisture equivalent shows the amount of moisture required to cause the soil to expand to its maximum.
6. *Shrinkage Limit.*—The shrinkage limit is the moisture content, expressed as a percentage of the dry weight, at which shrinkage ceases. That is, further loss of water does not increase shrinkage.

[1] Converse, Frederick J., "The Practical Use of Shear Test Data," a paper presented at the 36th Annual Convention, American Road Builders' Association, Mar. 7, 1939, at San Francisco.

Palmer, L. A., "Principles of Soil Mechanics," *Public Roads*, Vol. 19, No. 10, p. 195, December, 1938.

7. *Shrinkage Ratio.*—The shrinkage ratio is the ratio of the percentage of volume change due to loss of water from a soil to the percentage change in the moisture content of the soil, that is, volume change in percentage divided by moisture loss in percentage. A high shrinkage ratio indicates a soil that may be expected to shrink or swell considerably with small changes in water content.

8. *Porosity.*—The porosity of a soil is the volume of the pores in the soil divided by the volume of the soil particles plus the pores. The porosity before and after manipulating and compacting a soil shows the shrinkage that may take place in moving soil from excavations to compacted embankments. It also shows the void space that may become filled with water with consequent loss of stability.

9. *Voids Ratio.*—The voids ratio of a soil is the volume of voids in the soil divided by the volume of the soil particles. This factor gives information similar to the porosity test and in addition is useful in calculating the specific gravity of a soil. In a completely saturated sample of soil, let w equal the weight of water in the pores in percentage of the weight of dried solids, e the voids ratio, and G the specific gravity.
Then

$$G = \frac{100e}{w}.$$

Classification of Subgrades.—The classification of subgrades according to the character of the soil of which they are composed and their topographical location have long been the objective of the scientists who have been studying soil as a highway material. The classification that is shown in Table VII is primarily the work of the soils staff of the U.S. Public Roads Administration, although the laboratories of the state highway departments have contributed to the correlation of the data on soils tests with the service behavior of the road surfaces built on those soils. By employing a standard classification of this type the time and expense of soils testing by a highway laboratory are reduced materially below that which would be occasioned if each project had to be handled as a new case. The classification referred to above has not been universally accepted, but it is gaining in favor as it becomes understood; and although it will doubtless be revised and broadened, it affords an admirable starting place for the detailed study of soils as a highway material. In this connection the following statement is significant.

An effort is made to show (*a*) that the subgrade instead of the pavement really supports the wheel load, (*b*) that the manner in which the subgrade supports the wheel load depends upon its reaction to both load and climatic changes, (*c*) that these reactions depend upon the five

basic physical characteristics of soils, to wit: cohesion, internal friction, compressibility, elasticity, and capillarity, (*d*) that these physical characteristics control such important performances of subgrades as shrinkage, expansion, frost heaving, the settlement of fills, sliding in cuts, and lateral flow of soft undersoils, (*e*) that these physical characteristics are furnished by soil constituents easily identified in the laboratory, and (*f*) that subgrades may be arranged in definite groups according to the characteristics of the soil constituents[1] [see Table VII].

The Soil Profile.—The soil profile of the right-of-way of the highway is in reality a vertical cross-section along the center line of the proposed location or some other convenient reference line upon which are indicated the location and extent of each of the distinct soil types encountered on the project.

The data for the soil profile are assembled by examining the soil in its natural field condition wherever it is exposed in excavations, road cuts, and trenches for tile lines, supplemented by the result of borings made with the ordinary soil auger to depths sufficient to penetrate the layers of soil that are likely to have a bearing on the design of the road surface.

At the time of the field examination notations are made as to the texture, color, structure, consistency, compactness, cementation, and visible evidence of chemical composition. Samples are taken from each layer for laboratory examination.

The test holes are located with reference to the survey stationing so that the occurrence and extent of each type of soil can be indicated to scale on the soil profile map.[2]

Densification of Manipulated Soil.—The construction of fills on the highway, and frequently the subgrades for road surfaces in cuts also, requires the soil to be handled by machinery that involves loosening it and compacting it again after it has been placed in the fill or after the subgrade has been shaped. It has been found that soil that is compacted at a predetermined degree of wetness by a certain method, such as by means of a three-wheeled or sheep's-foot roller, will reach the most stable and permanent density. Experience seems to indicate that soils so densified will retain their stability for a long time. In fact

[1] HOGENTOGLER, C. A., A. M. WINTERMYER, and E. A. WILLIS, "Subgrade Soil Constants," *Public Roads*, Vol. 12, No. 4, p. 89, June, 1931.

[2] The detailed instructions for surveying and sampling soils in connection with highway improvement will be found in A.S.T.M. Tentative Standard Method D420-35T in the 1935 *Proceedings* of the society, p. 940.

the method is being used with success in the construction of earth-fill dams of great magnitude.

The key to success in this process is a method of determining the correct moisture content to employ in compacting the soil, developed by R. R. Proctor,[1] which is as follows:

Apparatus: The apparatus consists of a cylinder of 4 in. internal diameter and 4½ in. depth, mounted on a detachable base plate and fitted with a filling collar 2 in. high, a 5½-lb. cylindrical rammer with an end area of 3 sq. in., and a plasticity needle which is a rod of a cross-sectional area of a decimal part of a square inch, such as 0.01, 0.1, . . . 1 sq. in. The needle is mounted in a device that permits a direct reading of the force required to press the needle into the soil.

Procedure: A sample of soil passing the No. 10 sieve is moistened slightly and packed into the cylinder in three layers, each of which is tamped with 25 blows, the tamper falling 1 ft. for each blow. The sample is struck off to the top of the cylinder and weighed. The density is measured by forcing the plasticity needle into the soil at the rate of ½ in. per second and determining the pressure required in pounds per square inch of needle cross-section. The soil, or a sample of it, is oven dried, and the moisture content determined. By repeating with a slightly increased amount of moisture each time, a curve can be determined for that soil which will show the moisture content at which maximum density is obtained, that is, the maximum amount of soil by dry weight that can be packed into the cylinder.

Computations: The method of showing the results of the laboratory tests is indicated in Fig. 23, in which the upper curve shows the actual weight of the soil per cubic foot when compacted in the ring, including the contained moisture.

The plasticity of each test specimen having been determined, the engineer is provided with the data for specifying the moisture content at which the embankment shall be compacted and a means of checking the effectiveness of the rolling by use of the plasticity needle.

The embankment is then compacted at the predetermined moisture content, which is accomplished by building the fill in lifts of about 6 in., each of which is sprinkled to secure the correct moisture content. The plasticity is checked as the

[1] PROCTOR, R. R., "Fundamental Principles of Soil Compaction," *Eng. News-Record*, Vol. 3, No. 9, pp. 245 *et seq.*, Aug. 31, 1933.

work proceeds, by means of the Proctor needle. It is not always possible to use a three-wheeled roller on soil at the optimum moisture content, but the sheep's-foot roller can be used as a rule. Some gradings of soil are too harsh at the optimum moisture content for the sheep's-foot roller but can be handled with a three-wheeled roller.

Subgrades may sometimes profitably be tamped when wet to get a density similar to that secured in fills, but only the upper

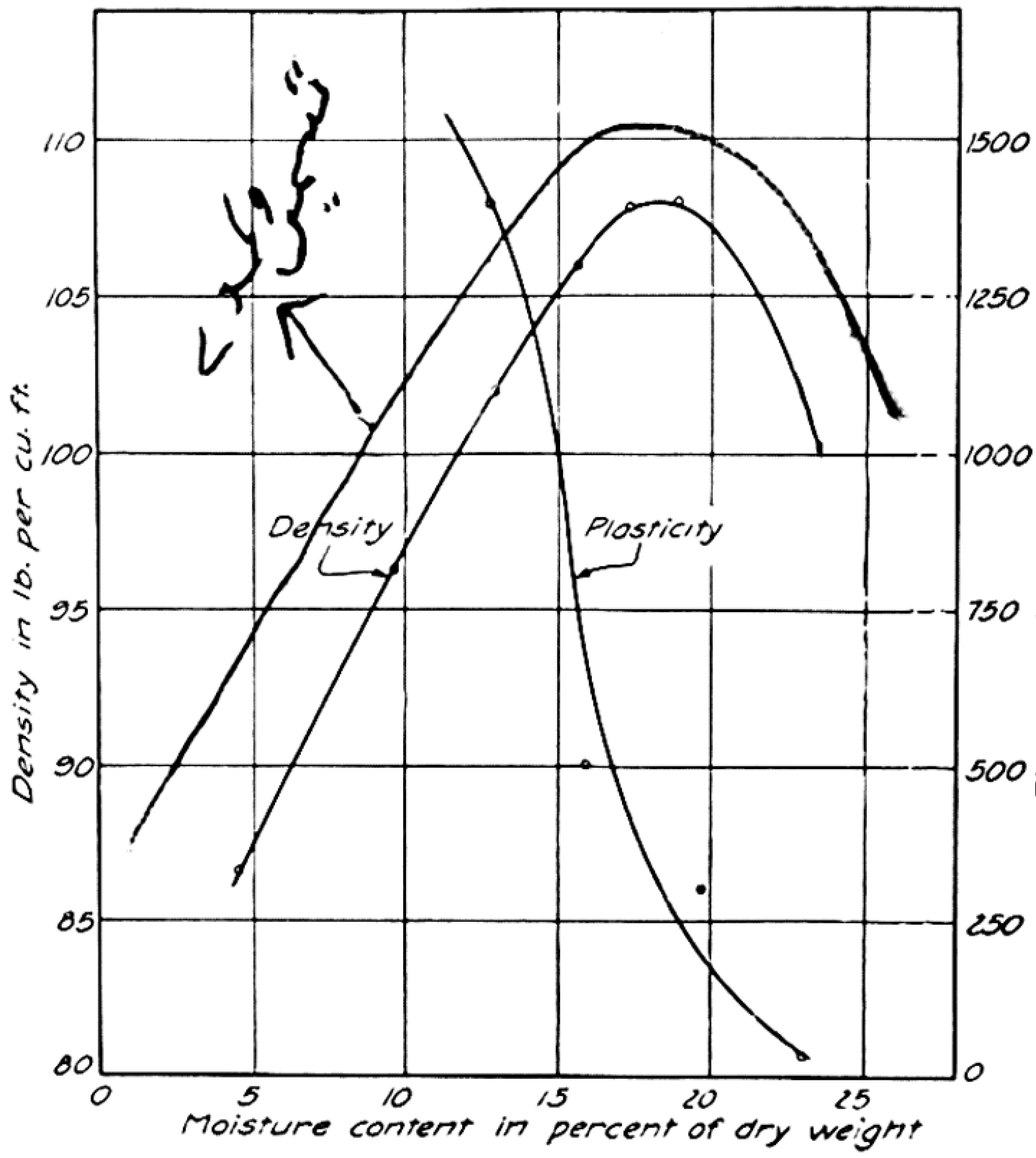

FIG. 23.—Illustrating the Proctor method of studying plasticity.

2 or 3 ft. need be tamped. In such cases, it is vital to provide against seepage of water to or rise of capillary water into the subgrade. In either case the beneficial effects of the tamping will be short-lived.

Soil as a Binding Agent.—Soil is frequently used as a binding agent or mortar to hold together materials that of themselves lack cohesion. Sandy soils, gravel, broken stone or slag, and similar granular materials are frequently used for soil-bound wearing surfaces, generally with little consideration of the suitability of the soil employed as a binder. Soil serves as a binding agent largely through the effect of the surface tension

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of the hygroscopic moisture held in the soil. For the retention of a moisture film it is essential that the void space in the mass be the minimum and the voids very small. This kind of mixture can be found in some localities but if not available can be secured by blending two or more soils and by thorough mixing and compacting.

The soil may be thought of as a mortar that fills the voids in the coarser material and serves to bind the material into a stable mass. For the best results the soil mortar ought to be fairly well graded; that is, it ought to approach the particle-size distribution shown by the Talbot curve in Fig. 21. The soil represented by curve 1, in that figure, is a well-graded one. In addition the soil should be cohesive and have low shrinkage, properties that are determined largely by the relative amounts of silt, clay, and colloids in the material but which must always be investigated by laboratory studies. The general specifications for soil mortars and a diagram showing the acceptable limits of grading are given in Chap. IX.

APPLICATIONS

In the design of structures, the load is determined and the dimensions of the load-carrying members of the structure are calculated on the basis of the known strength of the particular structural material to be used. A strictly analogous process can be followed in the design of a roadway surface, although data on the strength or, more properly, load-carrying capacity of soils are not readily available in handbook form. On the contrary, they must be established by tests on the materials actually to be employed in the construction, which is readily accomplished by methods previously outlined or referred to herein.

Subgrade Loads.—The least definite factor in the problem of highway design is that of the load transmitted through the roadway surface to the subgrade. Theoretically the wheel load is transmitted through the roadway surface according to a theory of stress distribution developed by M. J. Boussinesq in 1885. The distribution of the pressures over the affected area, if represented graphically, has the appearance of an inverted helmet and is in consequence sometimes referred to as the "helmet of stress." The Boussinesq equation applicable to the

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TABLE VII.—SUMMARY OF CHARACTERISTICS OF SUBGRADE GROUPS

P.R.A. group	General character of the soil	Constants for the soil[1]	Treatment required in construction
A-1	Grading: Material retained on No. 10 sieve not more than about 50 per cent. Soil mortar, that fraction passing No. 10 sieve, to consist of clay, 5 to 10 per cent; silt, 10–20 per cent; total sand, 70 to 85 per cent; coarse sand, 45 to 60 per cent. Average efficient size approximately 0.01 mm., and uniformity coefficient greater than 15. Well-graded material, coarse and fine, excellent binder. Highly stable under wheel loads, irrespective of moisture conditions. Functions satisfactorily when surface-treated or when used as a base for relatively thin wearing courses. Sandy loams, sandy clays	Liquid limit not less than 14 or greater than 25; plasticity index seldom greater than 8; shrinkage limit seldom less than 14 or greater than 20; and centrifuge moisture equivalent not apt to be greater than 15	Wearing course sufficient. Drainage to prevent frost heave when ground-water level is high. Responds to tile drains
A-2	Grading: Not less than about 55 per cent of sand in soil mortar. Sandy loams, loams, silts. Coarse and fine materials, inferior binder. Highly stable when fairly dry. Apt to soften at high water content caused either by rains or by capillary rise from saturated lower strata when an impervious cover prevents evaporation from top layer	Liquid limit generally not less than 14 or greater than 35; a plasticity index of zero with a significant shrinkage limit or a plasticity index greater than zero and less than 15 with or without a significant shrinkage limit; centrifuge moisture equivalent not greater than 25	Surface treatment by oiling to prevent softening of binder from above. Drainage to prevent frost heave and softening of binder from below. Load distribution through moderately thick non-rigid or thin rigid courses
A-3	Grading: Effective size not likely to be less than 0.10 mm. Sands, especially beach sand. Coarse material only, no binder. Lacks stability under wheel loads but unaffected by moisture conditions. Furnishes excellent support for flexible pavements of moderate thickness and for relatively thin rigid pavements	Liquid limit not appreciably greater than 35; no plasticity index, no significant shrinkage limit; centrifuge moisture equivalent less than 12. Ability of sands to resist sliding when wet indicated as follows: Liquid limits of 10–14 signify beach and other rounded sands which slide easily; liquid limits of 30–35 indicate rough angular particles which do not slide easily. In addition, liquid limits when lower than field moisture equivalents indicate materials that flow under partial saturation; when equal to field moisture equivalents, liquid limits indicate average sands which flow under full hydrostatic uplift. Liquid limits greater than field moisture equivalents indicate rough-grained sands which flow only	Coarse materials. Subgrade treatment by admixture of binder or light tars and substantial wearing course. Otherwise moderately thick non-rigid or thin rigid courses. Drainage not required

		when in a state less consolidated than that represented by field moisture equivalent	
A-4	Silt soils without coarse material and less than 55 per cent sand and with no appreciable amount of clay. Apt to absorb water very readily in quantities sufficient to cause rapid loss of stability even when not manipulated. When dry or damp, presents a firm riding surface which rebounds but very little upon removal of load. Apt to cause cracking in rigid pavements due to frost heaving and failure in flexible pavements due to low support	Liquid limit seldom less than 20 or greater than 40; plasticity index not greater than those indicated by curve 3; shrinkage limit not likely to be greater than 25; centrifuge moisture equivalent approaching those indicated by curve 10 between 12 and 50; when greater than liquid limit indicates varieties of soils inclined to be especially unstable in presence of water; field moisture equivalent equal to or somewhat greater than those indicated by curve 11 with a maximum of about 30. Increase in expansive properties generally indicated when shrinkage limits exceed 20 and approach those represented by curve 6; especially likely when field moisture equivalent exceeds centrifuge moisture equivalent	When naturally drained or when artificial drainage is possible: thick macadam or concrete pavement of medium thickness (not less than 8–6–8). Subgrade treatment by admixture of coarse constituents permits reducing thickness of macadam. Oiling combined with subgrade treatment may further improve quality. When there is a high ground-water level, and drainage is not possible: macadam unsuitable. Thick concrete pavement (not less than 9–7–9), crack control and reinforcement. Oiling not promising because water comes from below. Subbase may be beneficial for reducing frost effect
A-5	Same as for Group *A*-4, wet. Generally less than 55 per cent of sand. Less favorable construction material than *A*-4	Liquid limit usually greater than 35; plasticity index seldom greater than those indicated by curve 3; centrifuge moisture equivalent greater than 12, often lying between curves 9 and 10; not likely to waterlog. (Exceptions occur.) Shrinkage limit generally greater than 30 and greater than 50 for very undesirable members of this group. May approach values indicated by curve 6 for silts containing peat and approach those indicated by curve 7 for soils containing either diatoms or mica in appreciable amount. Field moisture equivalent approaching those indicated by curve 12 for silts containing peat in appreciable amount and those indicated by curve 13 for highly elastic soils containing mica or diatoms in appreciable amount. Kaolins, representing good binders, are members of group possessing relatively high plasticity indices and low field moisture equivalents	Similar to Group *A*-4 but furnishes highly elastic riding surfaces with appreciable rebound upon removal of load even when dry. Elastic properties interfere with proper compaction of macadams during construction and with retention of good bond afterward

TABLE VII.—SUMMARY OF CHARACTERISTICS OF SUBGRADE GROUPS—Continued

P.R.A. group	General character of the soil	Constants for the soil[1]	Treatment required in construction
A-6	Clay soils (30 per cent or more of clay) without coarse material. In stiff or soft plastic state, absorb additional water only if manipulated. May then change to liquid state and work up into the interstices of macadams. Furnish firm support essential in properly compacting macadams only at stiff consistency. Deformations occur slowly, and removal of load causes very little rebound. Shrinkage properties combined with alternate wetting and drying under field conditions are apt to cause cracking in rigid pavements	Liquid limit usually greater than 35; plasticity index approximately represented by curve 4; shrinkage limit not likely to be appreciably greater than that indicated by curve 5; centrifuge moisture equivalent test generally productive of waterlogging, likely to lie between curves 9 and 10; field moisture equivalent seldom exceeding those indicated by curve 11 but may be appreciably less for certain colloidal soils. Volumetric change generally greater than 17	Distinguished by state of soil whether impermeable (homogeneous) or permeable (full of cracks and root holes). Homogeneous state: ample load distribution by thick macadam or rigid pavement Degree of required load distribution depends on degree of softness. Surface treatment (oiling or screenings or both) prevents material from working into non-rigid base course. Crack control for reducing effect of unequal shrinkage. Permeable state, drainage feasible—macadam or rigid type. Subgrade treatment by mechanical manipulation under traffic increases stability. Permeable state, drainage not feasible—very strong macadam or rigid type with crack control. Reinforcement desirable. Subgrade treatment by admixture of coarse material on subbase for reducing frost heave. In fills—mechanical manipulation by traffic very beneficial. Also subgrade treatment by admixture of coarse constituents. Place fill in dry season. Springs entering base from below should be piped away. Treatment by oiling may reduce danger of saturation from above
A-7	Similar to Group *A*-6; seldom contains less than 30 per cent clay. When moist, deforms quickly under load and rebounds appreciably upon removal of load. Thus, lacks firmness in support, similar to subgrades of Group *A*-5. Alternate wetting and drying under field conditions lead to even more detrimental volume changes than in Group *A*-6 subgrades. May cause concrete pavements to crack before setting and to crack and fault afterward	Liquid limit usually greater than 35; plasticity index varies between those indicated by curves 3 and 4; shrinkage limit generally varies between those indicated by curves 5 and 6; centrifuge moisture equivalent varies between those indicated by curves 9 and 10; waterlogging in centrifuge test may not occur even at very high moisture equivalents. Field moisture equivalent greater than those indicated by curve 11. Relatively low shrinkage limits with high field moisture equivalents indicate presence of colloidal organic matter. Relatively high shrinkage limits indicate possibility of frost heave	Surface treatment by mechanical manipulation for preventing unequal expansion and by application of tar paper for preventing expansion beneath fresh concrete. Otherwise treat them like soft, homogeneous Group *A*-6 subgrades

TABLE VII.—SUMMARY OF CHARACTERISTICS OF SUBGRADE GROUPS—Continued

P.R.A. group	General character of the soil	Constants for the soil[1]	Treatment required in construction
A-8	Very soft peat and muck incapable of supporting a road surface without being previously compacted or displaced by a fill	Liquid limit greater than last; plasticity index generally less than those indicated by curve 3; shrinkage limit indicated approximately by curve 6; centrifuge moisture equivalent between curves 9 and 10; field moisture equivalent likely to be greater than those indicated by curve 12. Waterlogging in centrifuge test is characteristic of mucks containing clay and colloids, whereas very high equivalents without waterlogging are characteristic of peat not more than slightly decomposed	Fill on top of soft ground, according to Michigan and Minnesota practice. Pavement requires "beam" strength, ample crack control, and reinforcement

[1] Curves referred to in this column are given in Fig. 24.

case of a concentrated load applied at a "point" on the surface is

$$Z_z = -\frac{(n-2)P}{2\pi}\frac{\cos^n\theta}{h^2}, \tag{3}$$

where

Z_z = the vertical component of the portion of the converted (at a point) load that is transmitted through the wearing surface to any area on the upper surface of the subgrade, in pounds per square inch.

h = the thickness of the wearing surface, in inches.

P = the superimposed load, in pounds.

θ = the angle between the vertical axis and the vector from the area to the center of the subgrade area for which Z_z is being calculated.

n = an experimental constant dependent upon the relative stiffnesses of the roadway surface and soil of the subgrade.

TABLE VIII.—CLASSIFICATION OF SUBGRADES

Sample number	Particle-size distribution					Liquid limit	Plastic limit	Plasticity index	Centrifuge moisture equivalent	Shrinkage limit	Shrinkage ratio	U.S.P.R.A. subgrade group based on the analysis in preceding columns	Prevailing type of soil
	Retained on 2.0 mm. gravel, per cent	2.0–0.5 mm. sand, per cent	0.05–0.005 mm. silt, per cent	0.005–0.000 mm. clay, per cent	Less than 0.001 colloids, per cent								
80	0.4	71.1	14.6	13.9	7.5	21.4	13.3	8.1	14.1	15.9	1.84	A-2	Sandy loam
38	12.1	44.4	20.5	23.0	13.1	38.1	13.9	24.2	23.8	10.2	2.03	A-6	Sandy clay loam
3	0.1	9.4	57.8	32.7	12.1	44.3	19.1	25.2	31.7	16.1	1.82	A-7	Silty clay
150	1.9	88.8	4.7	4.6	2.4	....	....	....	7.9	18.1	1.69	A-3	Sand
114	0.1	30	43	26	13.0	34.7	16.4	18.3	20.2	16.4	1.84	A-4	Clay loam

When $\theta = 0$, Z_z becomes infinity according to Equation (3), which is due to the fundamental assumption of a "point" load upon which Equation (3) is based. A wheel load is applied over a finite area, and measurements show that the distribution of the wheel load to the upper surface of the subgrade follows the pattern indicated by Equation (3) except for the case when $\theta = 0$. Here, because the load is applied to the roadway surface over a finite area, the pressure on the surface of the subgrade is of finite magnitude.

By selecting suitable values for n according to the relative stiffnesses of the roadway surface and subgrade, as determined by experiment, Equation (3) may be employed to compute the pressure on the surface of the subgrade, *except in the region immediately under the wheel load*, where it must be estimated by continuing the distribution curve on the basis of judgment guided by experimental results, which, however, are quite definite in their indications.

Supporting Strength of Subgrade.—The analysis of the pressure on the subgrade from the traffic on the wearing surface,

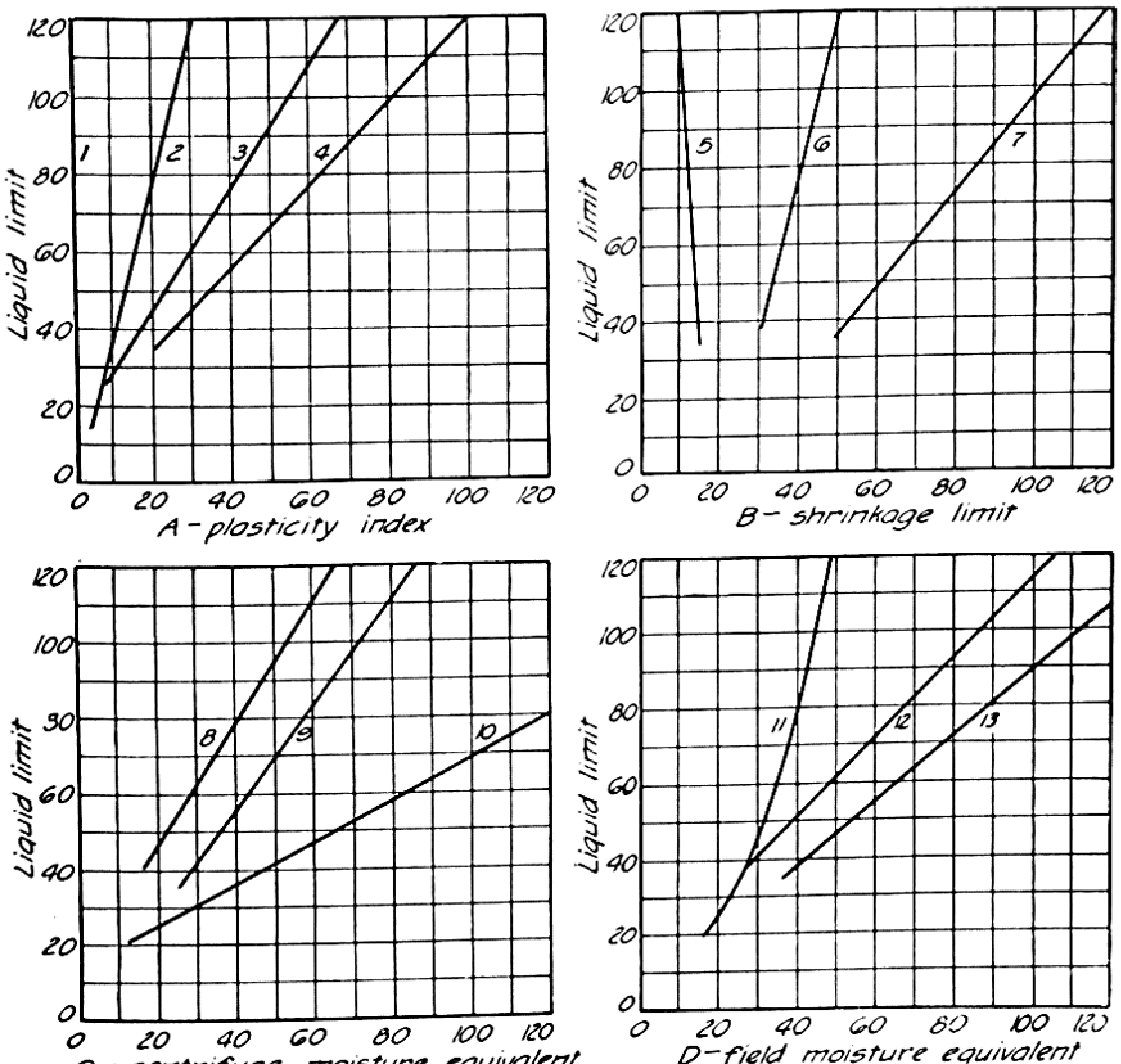

FIG. 24.—Illustrating the properties of soil given in Table VII.

as outlined in the foregoing, will afford a definite basis for providing a subgrade of the required strength. This involves two separate design problems:

1. The design of the drainage system required to insure the elimination of free water in the subgrade or under the subgrade for a depth of several feet.

2. The construction of the subgrade and embankments of such combination of materials and so manipulated that, under the least favorable weather conditions, it will have the required supporting strength.

The drainage system is designed after a study of the soil profile (page 98) to determine if there are water-bearing seams or strata underlying the roadway and after consideration of the laboratory tests on the several soils encountered, especially the data on porosity, voids, and particle-size distribution. The methods of drainage available for each condition were discussed in Chap. III.

The required supporting strength for the contemplated type of road surface being known, the soil of the right-of-way is tested to determine whether or not at the optimum moisture content it has the required supporting strength. If it has not, the necessary treatment for improving it can be determined from the laboratory tests. Almost invariably this treatment consists in the addition of granular material (sand or sandy gravel, crushed stone, or similar material) to the soil in quantities shown by laboratory tests to be needed to afford the desired supporting strength.

It happens frequently that the cost of securing the desired stability with the material available is excessive, and if so it may be advisable to increase the thickness of the roadway surface or change the design to a stiffer type so as to achieve the needed load-carrying capacity without treating the subgrade soil. The drainage would, of course, be installed in any case.

CHAPTER V

ECONOMICS OF HIGHWAY GRADES

Longitudinal grades constitute the most nearly permanent feature of surfaced highways, and a complete analysis of the factors governing this feature of design is therefore the first step in developing a balanced design for the improvement of a highway.

Basis of Design of Grades.—In railroad location the longitudinal grades are established almost wholly with relation to the characteristics of the traffic that is anticipated on the line. In the highway field the road builder and the operators of the traffic are entirely disassociated, and the highway department is not directly accountable for the cost of operation of the vehicles on the highways. Accordingly, the highway engineer has too often based his design of grades on the physical characteristics of the site and the cost of construction of the proposed improvement without any adequate consideration of the traffic. Recognition of the factors involved in providing for economical utilization of the highways is the outstanding development in present-day road management. The effects of grades, distance, curvature, and type and condition of road surface on the cost of vehicle operation and the safety of the traveling public are gradually becoming recognized as basic considerations in road design.

The designing of longitudinal grades involves the consideration of the *characteristics of traffic* and of certain physical factors associated with the site, such as *drainage*, *access to adjacent property*, *appearance*, and *safety*. Maximum grades will generally be established with a view to facilitating the movement of traffic. Grades below the maximum rate will be fixed with a view to keeping the earthwork costs as low as possible, after due consideration of the drainage, safety, and aesthetic features of the site.

Characteristics of Motor Vehicles.—Automobiles are designed with a view to securing mobility and that quality designated as

"performance," which really means that the engine has the power to produce a high rate of acceleration. The full power of the engine is seldom used on level roads, being in reserve (Fig. 25) for use on bad roads, on hills, and for acceleration. The engine, being of the throttle-governed, four-cycle type, is most efficient at full throttle at a speed selected by the designer (Fig. 26). When ascending hills that are not so steep as to retard the speed too much, the engine is somewhat more efficient

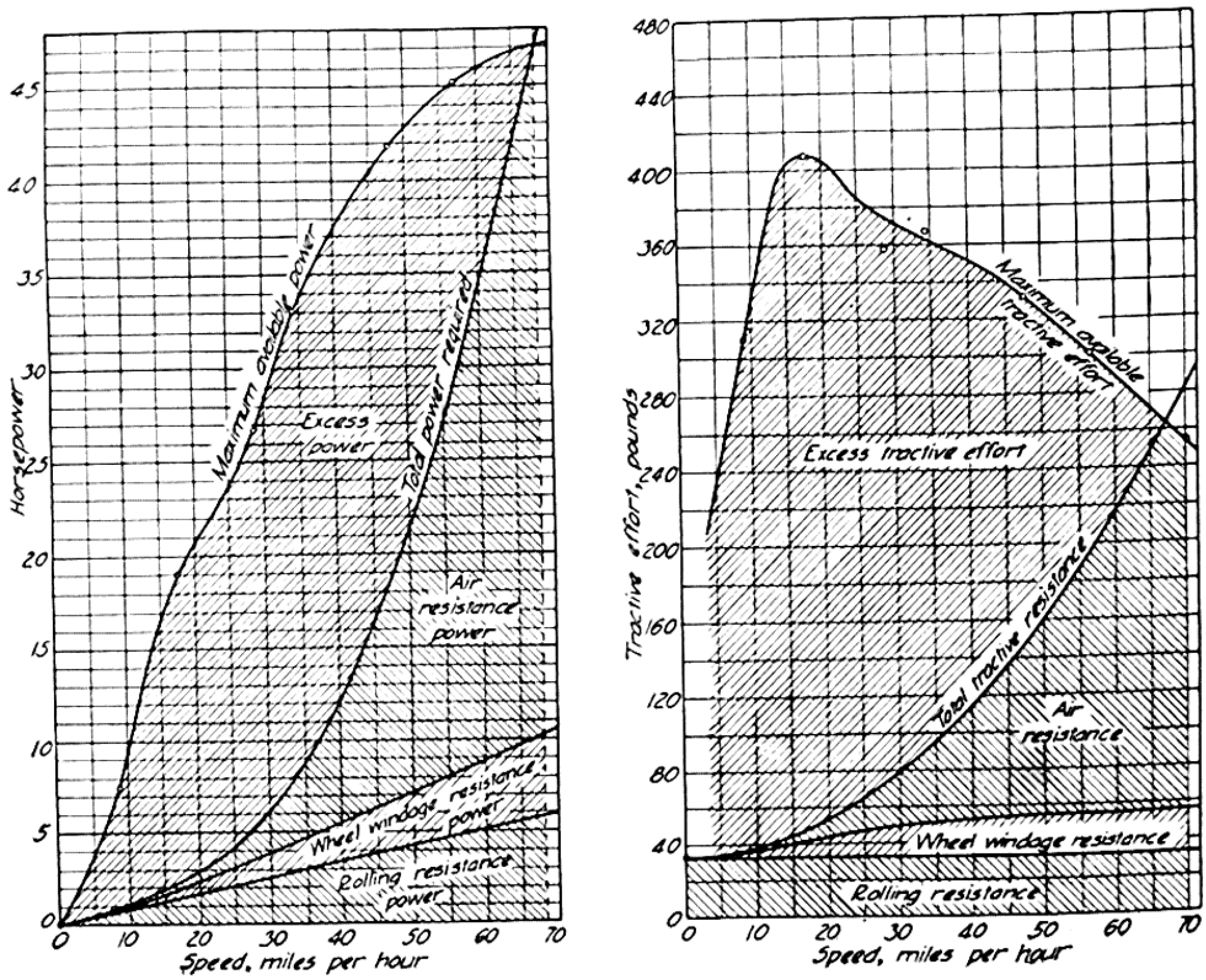

Fig. 25.—Showing power characteristics of automobiles.

than at the same speed on level roads, because the gravity component that must be overcome on the hill, added to the tractive resistance, gives a favorable load. Commercial vehicles such as trucks and busses are sometimes provided with excess engine power, as are automobiles, but the heavier commercial vehicles do not have great reserve power and depend upon the use of the change gears to a greater extent than do automobiles. Like the automobile engine, the commercial vehicle engine, whether of the ordinary or the Diesel type, is most efficient under full load at a certain speed, but full load is reached on lower rates of grade than those which give full load to the automobile engine. Up to a certain rate of grade, a hill creates a condition favorable

to the economical operation of a gas-engine driven vehicle. The exact rate of ascending grade that is the upper limit of this favorable operating condition depends upon the power of the engine and the gear ratios or, more properly speaking, upon the tractive effort that can be exerted by the driving wheels. This is generally expressed in terms of tractive effort, in pounds per ton of weight of vehicle.

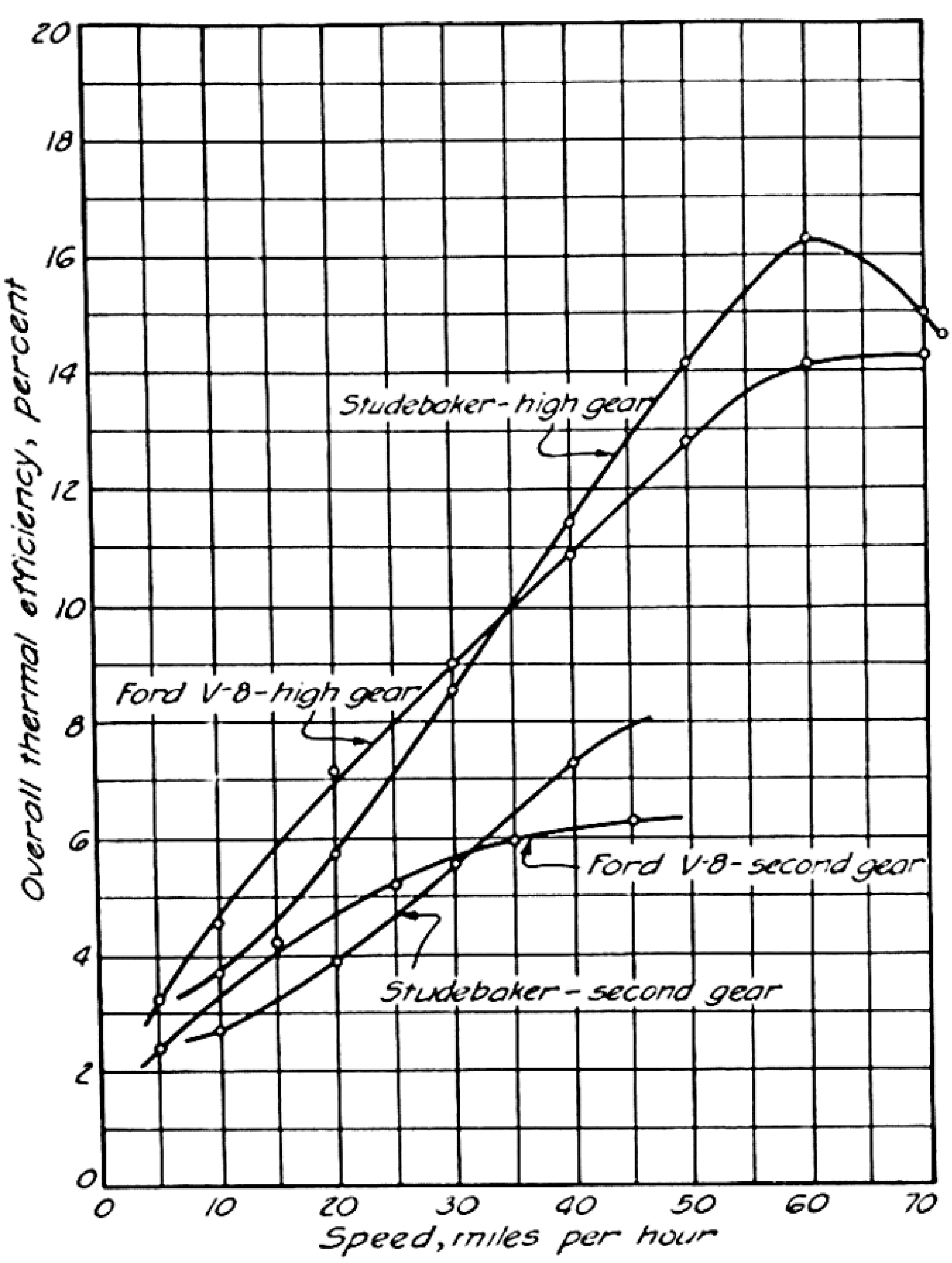

FIG. 26.—Showing relation between efficiency of automobiles and gear used.

The tractive effort in high gear may be insufficient to permit the vehicle to operate under certain conditions of road surface or grade, and change gears are provided to take care of this situation. But operation of the vehicle when driven through the change gears is less economical than when it is driven in high gear, because of the additional friction and lower engine efficiency (Figs. 27, 28). Therefore, when there is a change in gears during the ascent of a hill, power waste is introduced.

Plus Grades.—The maximum economical rate of grade for an automobile to ascend is one that will permit ascent in high gear at, or near, a constant road speed of 50 to 60 miles per hour. This rate of grade, when determined, is the maximum economical plus grade for an automobile highway. For the automobile of 1936 design this is an 8 per cent grade (Fig. 29).

Likewise, the maximum economical grade for a commercial vehicle to ascend is one that will permit the ascent at the proper speed without shifting gears. For the freight trucks of 1936

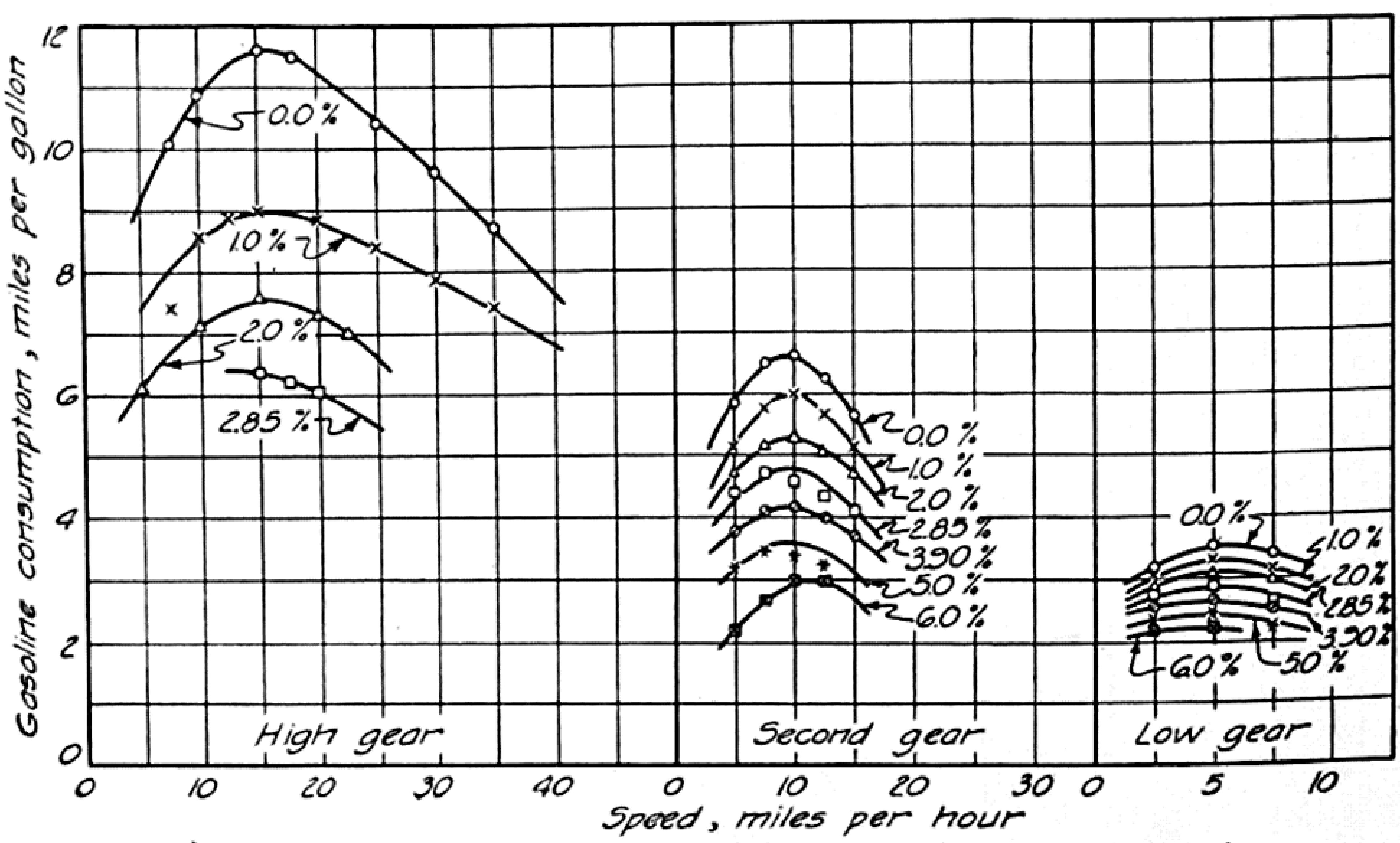

FIG. 27.—Showing relative gasoline consumption of trucks in various gears ascending various grades.

this is about a 3 per cent grade. The grade of a highway can never anywhere nearly be correct for both classes of traffic, nor can it be correct for all classes of commercial vehicles. A compromise of all the divergent requirements must be adopted, and that can be accomplished acceptably only when the exact effect of grade on the cost of transportation is thoroughly understood (Fig. 30).

If the average tractive effort, in high, intermediate, and low gear, of the vehicles that make up the traffic on a highway has been determined, it will then be possible to adapt the highway design to the vehicle, so that grades will be the most economical, in view of the known volume of each class of vehicles that uses the road.

Tractive effort is available to overcome tractive resistance and the effect of grade. If, from the maximum tractive effort of a vehicle in high gear in pounds, the tractive resistance for

that vehicle in pounds is subtracted, the remainder, divided by 20, is the maximum economical grade in percentage for that vehicle, provided momentum effects are not taken into account. The rate of grade thus determined may require so much work that the cost will not be justified by the volume of traffic on the highway, but an estimate of the situation must begin with the theoretical maximum grade.

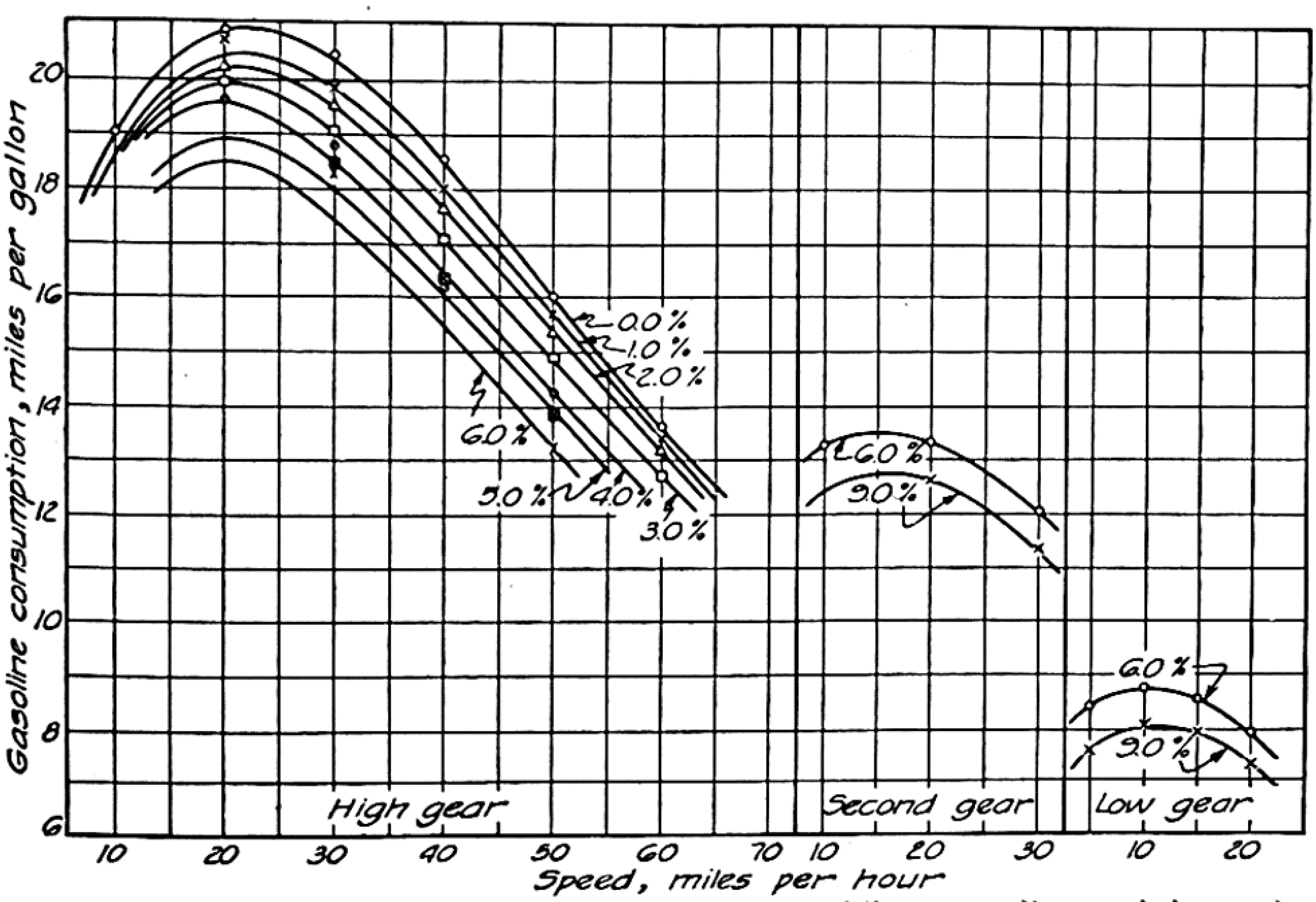

FIG. 28.—Showing gasoline consumption of automobiles ascending and descending various rates of grade.

The maximum economical grade for ascending vehicles may be calculated as follows:

$$P_p = \frac{T}{20} - \frac{R}{20}, \tag{1}$$

where

P_p = maximum economical plus grade (vehicle ascending), in feet per 100 ft. of road.

T = the maximum tractive effort of the engine at the desired speed in high gear, in pounds per ton of weight of vehicle and load.

R = the tractive resistance of the vehicle on the proposed type of road at the desired speed, in pounds per ton of weight of vehicle and load.

Momentum Effects.—The automobile readily lends itself to a method of operation that takes advantage of momentum

effects in hill climbing. This is merely another way of saying that hill climbing is facilitated by "taking a run" for the hill. If the vehicle slows down when ascending a hill, a certain amount of kinetic energy is transformed into the work of climbing the hill. Commercial vehicles have the same mobile characteristics as automobiles but to a lesser extent. Assume that the speed

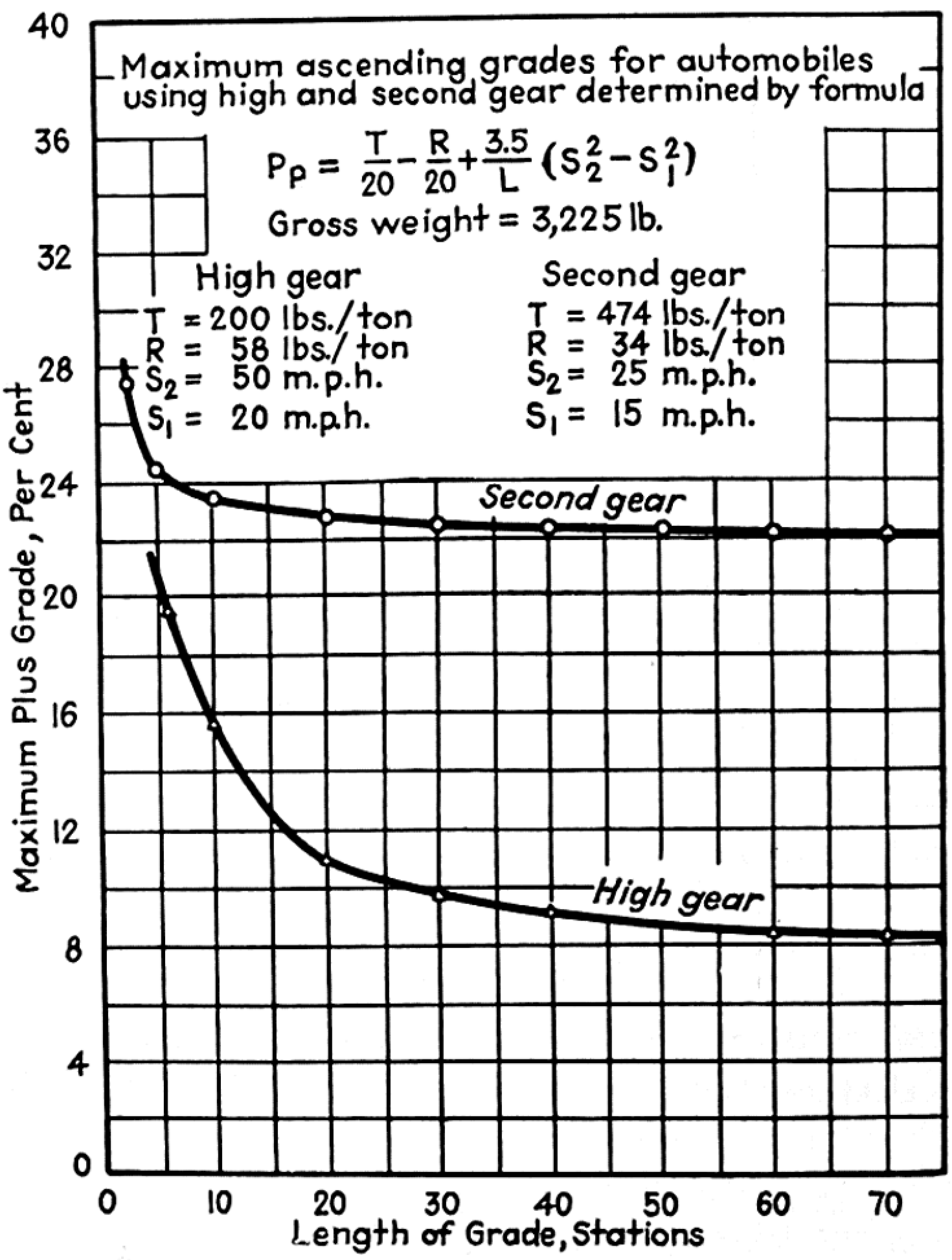

FIG. 29.—Showing efficient plus grades for automobiles.

of the vehicle at the beginning of the ascent is V_1 ft. per second and that when the ascent has been completed the speed has been reduced to V_2 ft. per second.

The loss in kinetic energy in foot-pounds is

$$E_k = \tfrac{1}{2}m(V_1^2 - V_2^2) + \tfrac{1}{2}I(w_1^2 - w_2^2). \qquad (2)$$

The quantity $\frac{1}{2}m(V_1^2 - V_2^2)$ will be recognized as the change in kinetic energy of translation due to the change in speed.

The quantity $\frac{1}{2}I(w_1^2 - w_2^2)$ is the corresponding change in kinetic energy of rotating parts, which cannot be calculated readily but has been determined experimentally. For ordinary computations involving automobiles, it may be taken as 5 per cent of the kinetic energy of translation; and for commercial vehicles, 10 per cent. The total change in kinetic energy takes place in the length of the hill, which should be measured along the actual profile, but which for the purposes of the highway engineer may be taken as the distance given by the profile stationing, since the error thus introduced is negligible. The average force contributed by the change in kinetic energy of an automobile over the length of road in which the speed changed from V_1 to V_2 is therefore

$$\text{Average force} = \frac{\frac{m}{2}(1.05)(V_1^2 - V_2^2)}{L}. \tag{3}$$

This may be written

$$\text{Average force} = \frac{70}{L}(S_1^2 - S_2^2), \tag{4}$$

where S_1 and S_2 represent the speeds in miles per hour corresponding to V_1 and V_2.

Table IX.—Characteristics of the Typical Automobile

Speed range, miles per hour	Tractive effort on the several gears, pounds per ton		
	High	Second	Low
40–60	200	475	650

Relative fuel mileage on the various gears. $\left\{\begin{array}{l}\text{High gear, 17 mi. per gallon}\\\text{Second gear, 12 mi. per gallon}\\\text{Low gear, 7 mi. per gallon}\end{array}\right.$

Characteristics of Typical 3-ton Commercial Vehicle

Speed range, miles per hour high gear	Tractive effort on the several gears, pounds per ton		
	High	Second	Low
25–45	65–100	90–150	200

By adding the grade effect of momentum as given in Equation (4) to Equation (1) there is obtained

$$P_p = \frac{T}{20} - \frac{R}{20} + \frac{3.5}{L}(S_1^2 - S_2^2). \tag{5}$$

In any location where density of traffic, topographical features, and traffic regulation permit safely taking advantage of momen-

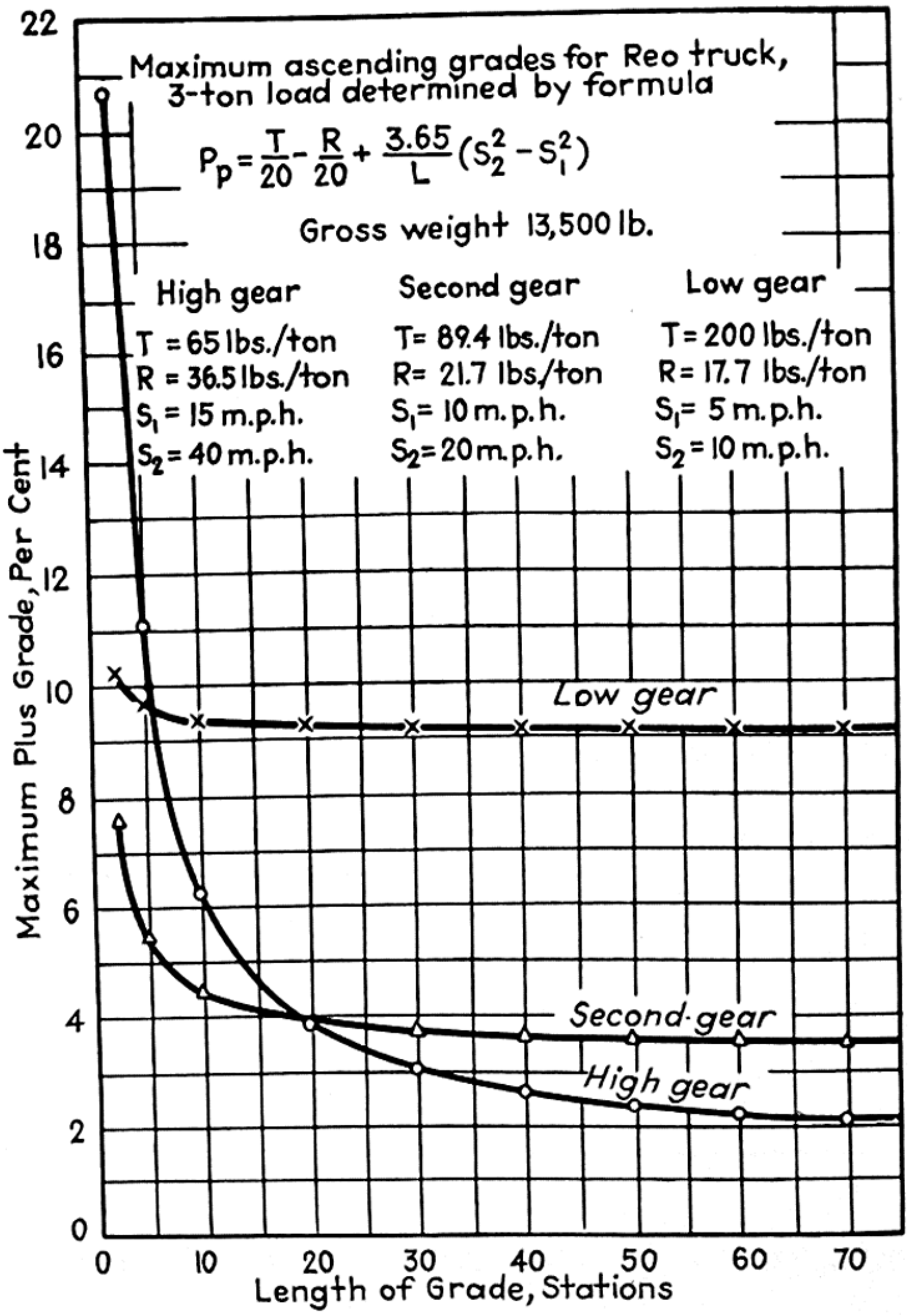

FIG. 30.—Showing efficient plus grades for trucks.

tum effects, the maximum grade for automobiles is determined from Equation (5). It will be noted that the maximum grade will decrease with L, for any assumed limits of speed.

Where it is apparent that momentum effects cannot be realized by the bulk of the traffic, the maximum grade can be determined from Equation (1).

Minus Grades.—When a vehicle is descending a hill, power from the engine will be used to maintain speed if the rate of grade is below a certain minimum. The vehicle will coast at uniform speed without power from the engine when the rate of grade is just right, and it will accelerate without power from the engine if the rate of grade is great enough (Fig. 31).

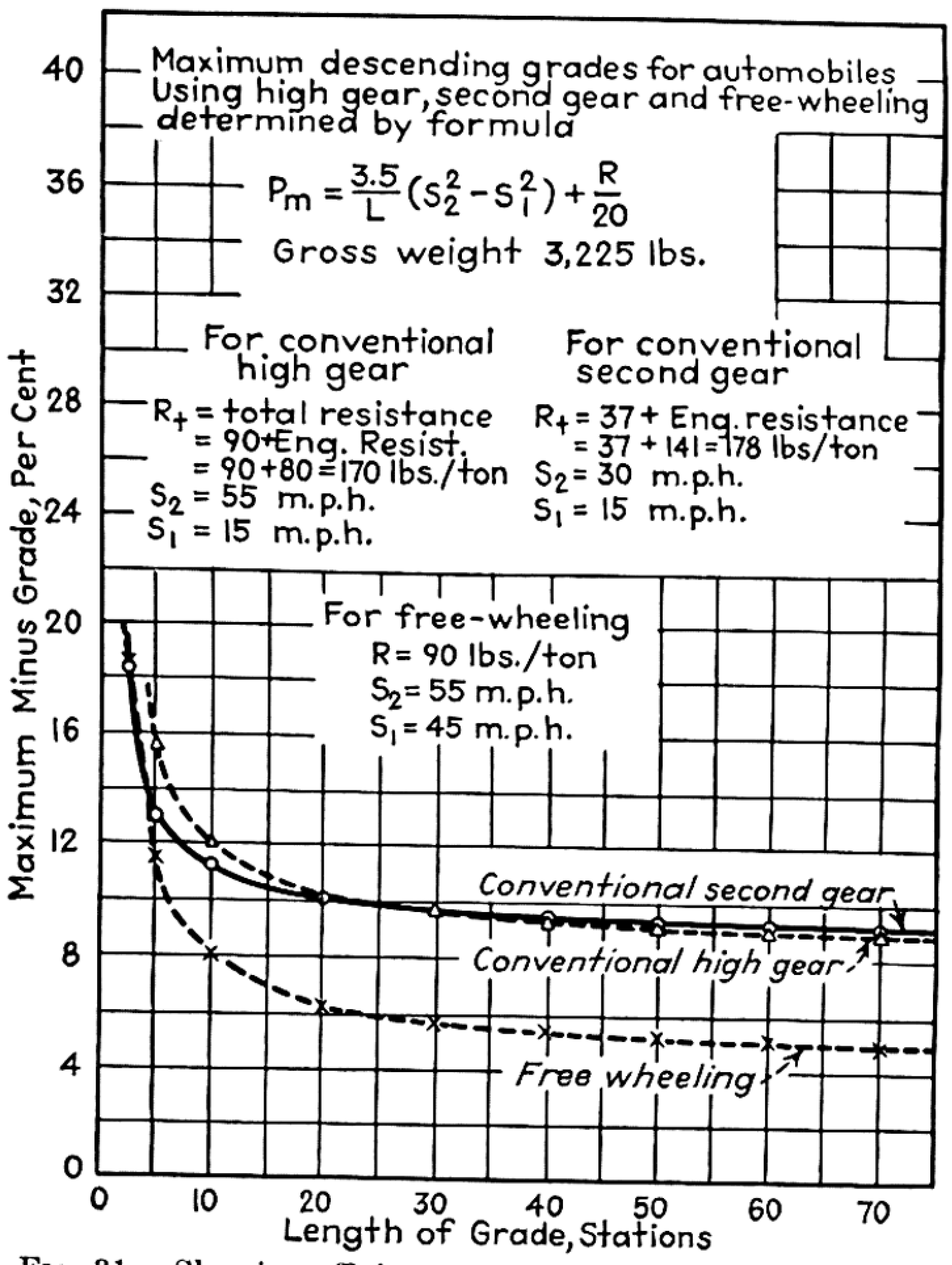

FIG. 31.—Showing efficient minus grades for automobiles.

The ideal grade for the descending vehicle is the one that will permit the vehicle to descend without using power or the brakes and without exceeding the speed fixed by regulations or considerations of traffic safety.

The grade P_m, which will permit the vehicle to coast at constant speed, is equal to $R/20$, where R is the tractive resistance for the particular road, vehicle, and speed, in pounds per ton.

By analogy with Equation (5), it will be apparent that if momentum effects are considered, this grade can be increased by

the grade equivalent of the momentum permitted, which for automobiles is $3.5/L(S_1^2 - S_2^2)$, in percentage of grade. Therefore,

$$P_m = \frac{R}{20}, \tag{6}$$

or

$$P_m = \frac{R}{20} + \frac{3.5}{L}(S_1^2 - S_2^2). \tag{7}$$

In this connection it should be noted that S_1 is used to designate the speed in miles per hour at the beginning of an ascent and at the end of the descent. The reason is obvious.

The maximum minus grade for economical automobile operation may be determined from Equation (6) if no momentum effect is to be considered and from Equation (7) if momentum effects are to be taken into account. The factors S_1 and S_2 will be determined by a process of reasoning identical with that involved in arriving at values for S_1 and S_2 to use in estimating maximum plus grades.

Grades for Commercial Vehicles.—The foregoing discussion has been based on the characteristics of automobiles, but precisely the same analysis applies to commercial vehicles, except that the ratio of the kinetic energy of rotating parts to that of translation is estimated to be 10 instead of 5 per cent. With that change the formulas for grades for commercial vehicles become

$$P_p = \frac{T}{20} - \frac{R}{20} + \frac{3.65}{L}(S_1^2 - S_2^2), \tag{8}$$

$$P_m = \frac{R}{20} + \frac{3.65}{L}(S_1^2 - S_2^2). \tag{9}$$

Rate-of-grade Diagrams.—The four equations developed in connection with plus and minus grades may be converted into diagrams that are convenient for use in establishing grades, by calculating the percentage grade for various lengths of hill, after inserting appropriate values for T, R, and L in the several equations.

In Table IX are average values of T for typical vehicles of 1936 design. Typical grade diagrams for automobiles are given in Figs. 29 and 30, and for one truck in Figs. 31 and 32, the data upon which they are based being indicated in the diagram. These are applicable only to grades for high-type road surfaces

in which the rolling resistance is no greater than indicated by the note on each diagram. Similar diagrams can be prepared for vehicles of characteristics different from those assumed and for road surfaces with a different rolling resistance.

Effect of Excessive Grades.—If a motor vehicle is driven up a grade, enough fuel must be used to lift the vehicle a height H. When the vehicle descends such a grade, all the potential energy will be utilized if the grade is not too steep; but if the rate of grade is in excess of the economical maximum for descending traffic, the potential energy cannot all be utilized because of the necessity for the use of the brakes to hold the speed to a safe rate. Moreover, the ascent of the grade will be possible only by using a gear other than high if the rate of grade is excessive, and consequently more fuel is used in lifting the vehicle than would have been required for an equal height on a rate of grade equal to or below the economical maximum. This is due to added friction losses and changed engine efficiency when reduction gears are used. Let x be the gallons of gasoline required to ascend a grade in high gear, and y the gallons required to ascend an equal height of grade in a lower gear; then $M = y/x =$ the relative fuel consumption on the two gears. The fuel consumed in lifting the vehicle through the height H by the use of a gear other than high is the same as for lifting it a height MH by the use of high gear. To get this relation into terms applicable to highway grades, it may be expressed thus:

$$E = W(MH - h), \tag{10}$$

where

$E =$ the energy equivalent of the fuel consumed because of excessive grade, in foot-pounds.

$H =$ the total rise in feet as shown by the profile, which can be expressed in terms of an average grade P.

$h =$ the total rise, in feet, when the grade has been reduced to P_p.

$W =$ the weight of the vehicle, in pounds.

It will be apparent that in some cases h and H will be equal, and the rate of grade will be reduced by increasing the length of the hill, which may be accomplished in some cases without changing the length of the route.

Since $H = PL$ and $h = P_pL_p$, Equation (10) may be written

$$E = W(MPL - P_pL_p), \tag{11}$$

in which

L = the length of grade having the rate P and height H.

L_p = the length of grade having the rate P_p and height h.

With gasoline weighing 5.9 lb. per gallon, testing 19,000 B.t.u. per pound, and costing C dollars per gallon, and an engine having an over-all efficiency of 15 per cent, the cost of the energy represented by E may be computed as follows:

$$\text{Cost} = \frac{(C)(E)}{(19{,}000)(777.6)(5.9)(.15)} = \frac{W(MPL - P_pL_p)C}{(19{,}000)(777.6)(5.9)(.15)}. \qquad (12)$$

The annual traffic ascending the hill in millions of tons is represented by V_a.

The annual cost of the excess plus grade for the traffic V_a is

$$D_a = \frac{(C)(V_a)(1{,}000{,}000)(2{,}000)(MPL - P_pL_p)}{(19{,}000)(777.6)(5.9)(.15)}. \qquad (13)$$

Since this is an annual cost, the saving to traffic due to the elimination of the cost of excessive grades may be capitalized on an appropriate basis to ascertain what sum could justifiably be expended for grade reduction. It is probably unwise to extend the period of assumed benefit indefinitely because of the uncertainties that exist with reference to the kind and quantity of highway traffic in the future. If it is assumed that these benefits will surely continue for 25 years and that money can be borrowed at 4 per cent interest, then the justifiable expenditure for reducing a specific grade is the present worth of an annuity of D_a dollars running for 25 years. On that basis,

$$S_a = 24CV_a(MPL - P_pL_p), \qquad (14)$$

S_a = the justifiable expenditure, in dollars, for grade reduction to the appropriate economical plus grade P_p.

C = the cost of gasoline, in dollars per gallon.

V_a = the annual traffic, in millions of tons ascending the grade.

M = a factor representing the ratio of the fuel consumption for the gear used on the grade P to the consumption in high gear on the grade P_p.

Excessive Minus Grades.—In the nomenclature employed in the foregoing discussion a "plus" grade is one that a vehicle

ascends. Obviously then, any hill that is a plus grade for the traffic in one direction is a "minus" grade for the traffic in the opposite direction. The energy expended in raising the vehicle on grades of a rate in excess of that which can be ascended in high gear within prescribed speed limits has been evaluated in the preceding discussion; but since the most economical minus grade is always a lower rate than the economical plus grade, it remains to determine the justifiable expenditure for reducing the grade to any rate less than the economical maximum plus grade.

By a process of analysis identical with that employed in the discussion of plus grades, the justifiable expenditure for reducing grades for descending traffic can be shown to be

$$S_d = 24CV_a(P_pL_p - P_1L_1) \tag{15}$$

in which P_1 is any rate of grade greater than P_m, and L_1 is the length of the grade P_1.

Procedure in Establishing Grades.—A very casual inspection of the profile of a proposed improvement will indicate which hills need to be checked from the standpoint of maximum grades, and the length and average rate of grade on the existing profile can be jotted down. A grade diagram has presumably been prepared on the basis of the proposed type of improvement and the characteristics of the vehicles that comprise the traffic. The volume of automobile and commercial vehicle traffic that will probably use the road has also been estimated.

With these preliminaries out of the way, the correct plus grade for each class of traffic is noted on the profile for each hill where the existing grade is in excess of the economical maximum.

The sum that can be justifiably expended to reduce each grade to the theoretically correct rate for each class of traffic is then estimated from Equation (14). A rough estimate of the probable cost of the earthwork necessary to reach the proposed maximum grade is next prepared by establishing a preliminary grade line for the hills in question. If in any case the cost of earthwork thus estimated is materially in excess of the justifiable expenditure, a special study is made of that location with a view to relocation or some modification of the grade line that will reduce the cost of earthwork.

It is apparent that estimates of the value of grade reduction cannot be exact, but they do give positive information in most

of the projects of major importance. The final decision will be based on the financial considerations, the importance of safety and convenience, and sometimes the drainage and topographical situation (danger of landslips, probability of excessive snow accumulations, etc.). On secondary roads the financial consideration is likely to govern more frequently than on trunk-line roads.

The second step is to ascertain which grades are in excess of the theoretically economical minus grades and to lay trial grades on them on the basis of ordinary engineering considerations such as sight distance, appearance, and drainage, being careful not to make any of the proposed grades less than the theoretical minus. The saving to traffic by these grade changes is then estimated from Equation (15). This revision does not affect a saving to ascending traffic.

The designer is now in a position to judge as to the efficiency of his preliminary grade line. If the cost of earthwork is to be materially in excess of the value to traffic, there is every probability that the quantity of earthwork should be reduced by further study of the grades.

It should be noted that separate studies must be made for automobile traffic and commercial vehicle traffic. In rough country, this sort of analysis involves a great deal of work, but the possibilities of saving in earthwork costs justify the effort. In other cases, the study will be applied to only a few critical grades.

In the great preponderance of highway-design problems, the grades are fixed on the basis of the requirements for automobile traffic, but the tonnage of commercial vehicle traffic now frequently reaches an amount that will materially affect the financial situation. In some instances, also, it will be wise to go beyond the economic considerations to reduce delay to commercial vehicles and thus minimize congestion.

Applications to Typical Grade Reduction Problems.[1]—The following illustrations are intended to show how the foregoing theoretical principles, considered in connection with the results of measurements of gasoline consumption and related data, may be applied to typical grade reduction problems. The

[1] Based on data in a report by R. A. Moyer, "Motor Vehicle Power Requirements on Highway Grades," published in *Proc. 14th Ann. Meeting*, Highway Research Board, 1934, p. 177.

constants required for the solution of these particular problems were obtained by operating a group of vehicles on various rates of grades and determining the actual fuel consumption at each of a number of speeds and with the use of each of the change gears. For a complete solution of the problem there should be

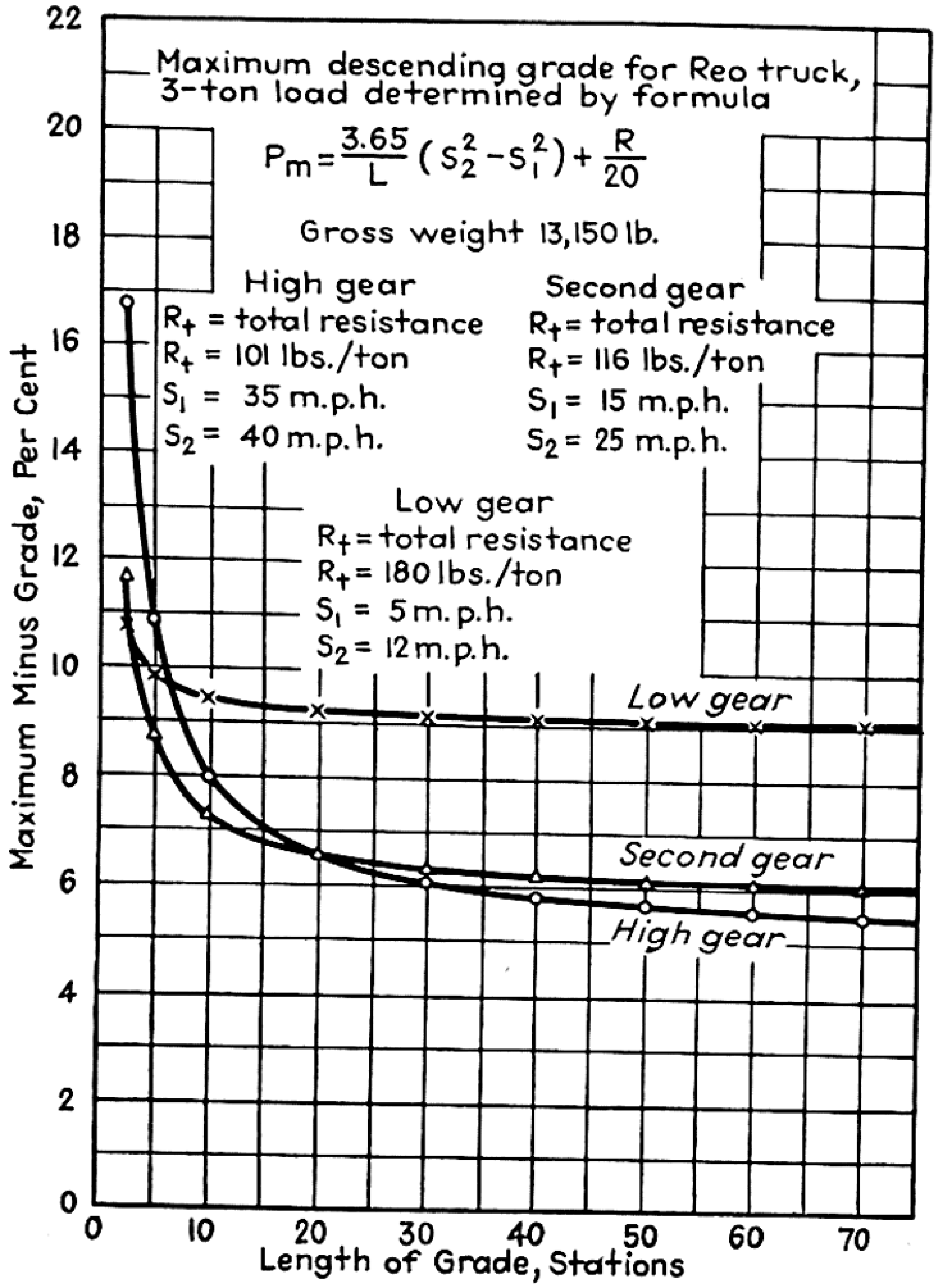

FIG. 32.—Showing efficient minus grades for trucks.

additional data on the operating characteristics of some of the heavier commercial vehicles. It is entirely feasible to make a similar analysis for any design problem.

No marked differences in the fuel consumption characteristics of various automobiles built during the past five years were observed in the course of these studies, and it is assumed that the fuel consumption rates obtained for the automobiles employed in this case are fairly typical of the automobiles of 1936 and will remain so until there are fundamental changes in automobile

engine design. The data obtained with the trucks have a limited application, for the reason that they indicate truck operating characteristics for only two of many types of trucks. The Graham represents the high-speed, 2-ton, platform type of truck, and the Reo represents the low-speed, moving van type, with a heavy body. Although the truck data used in the following illustrations are not universally applicable, they indicate characteristics of truck operation on grades that are fairly representative of a large group of trucks. The analysis also indicates a general relationship between savings made by grade reductions that may be attributed to truck operation as compared to those which may be attributed to passenger car operation.

In establishing highway grades, there are two general methods of grade reduction which are designated as Method I and Method II in Fig. 33. Method I is that in which the rate of grade is reduced by lengthening it without changing the total vertical rise. Method II is that in which the rate of grade is reduced by reducing the total vertical rise without increasing the length of grade. Two typical problems are illustrated by each method, the first involving the reduction of 1,600 ft. of a 9 per cent grade to a 6 per cent grade, and the second providing for the reduction of a 6 per cent grade 1,600 ft. long to a 3 per cent grade. The sections of highway involved in the computations were brought to the same length in each case by adding to the hill section the necessary length of 0.00 per cent grade. For simplicity, vertical curves were eliminated but this does not materially affect the comparative value of the results.

The gasoline consumption rates used in the analysis were the average for passenger cars as determined specifically for the purpose and the actual rates for a Graham and a Reo truck, also determined for this purpose. The volume of traffic was assumed to be 1,000,000 tons of automobile traffic per year in each direction and an equal tonnage of truck traffic. The maximum *plus* and *minus* grades over which the three types of vehicles could operate at assumed rates of speed in the various gears were determined from the curves in Figs. 29, 30, 31–35 inclusive. These curves were based on the actual operating characteristics of the vehicles, determined during operation on the road, and were applied in Formulas (5), (7), (8), and (9). The speeds, gears, and fuel rates used in the solution of the grade reduction problems are given in Table X.

The savings in gasoline due to the reduction of these grades, in gallons per year and in cost per year, are given in Table XI. Reduction of the grades resulted in savings for both automobiles and trucks in all cases except for the Reo truck on the I-*A* grade. In this last named case the operation on the 9 per cent grade with

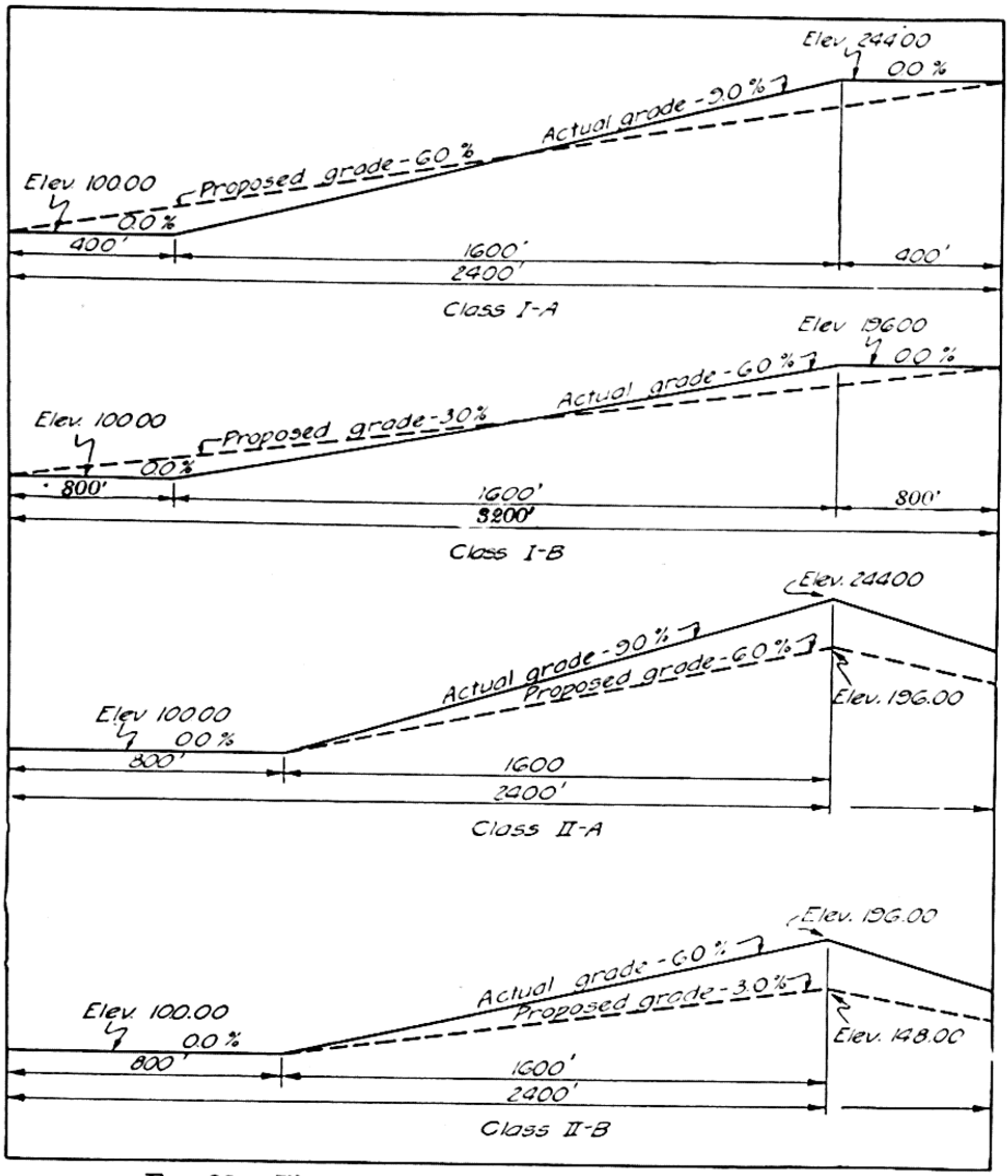

Fig. 33.—Illustrating calculation of economical grades.

the 0.0 grade approaches was cheaper than on a continuous 6 per cent grade of the same total length and the same total height. The reduction in height by Method II made possible larger savings than when Method I was used, because Method I contemplates no change in the total height of the hill.

In addition to the fuel savings, large time savings in truck operation resulted from the reduction in grades. A summary

of the time saved on the different grades for the two trucks is given in Table XII. For the Graham truck the time saved in the reduction of a 9 to a 6 per cent grade was about ten times as great as that obtained from reducing a 6 to a 3 per cent grade. For the Reo truck the time saving was greater when reducing a 6 to a 3 per cent than from a 9 to a 6 per cent grade. Since the passenger cars were assumed to operate at a constant speed of 40 miles an hour on all of the grades, no saving of time resulted in their operation on the reduced grades.

This analysis indicates the correct approach to a study of the economics of grade reduction. The data required can be secured in a few days by a series of rather simple observations on the operating behavior of vehicles typical of the traffic on a particular highway.

Experimental Coefficients.—In estimating the effect of various rates of grade upon the cost of operating self-propelled vehicles,

TABLE X.—ASSUMED SPEEDS, GEARS, AND GASOLINE CONSUMPTION ON DIFFERENT GRADES

Type of vehicle	Grade, per cent	Ascending				Descending			
		Gear	Speed, miles per hour	Gas consumption, gallons per ton-mile	Class of grade[1]	Gear	Speed, miles per hour	Gas consumption, gallons per ton-mile	Class of grade[1]
Passenger cars	0.00	High	40.0	0.034	I and II	High	40.0	0.034	I and II
	3.00	High	40.0	0.049	I and II	High	40.0	0.024	I and II
	6.00	High	40.0	0.064	I and II	High	40.0	0.014	I and II
	9.00	High	40.0	0.090	I and II	High	40.0	0.009	I and II
Graham truck (gross load 10,500 lb.)	0.00	High	20.0	0.015	I and II	High	20.0	0.015	I-*A*, II-*A*
	0.00	High	40.0	0.020	II-*B*	High	40.0	0.020	I-*B*, II-*B*
	3.00	High	25.0	0.034	I-*B*, II-*B*	High	40.0	0.009	I-*B*, II-*B*
	6.00	Second	20.0	0.060	I and II	High	40.0	0.0055	I and II
	9.00	Low	10.0	0.091	I-*A*, II-*A*	Low	10.0	0.027	I-*A*, II-*B*
Reo van (gross load 13,150 lb.)	0.00	High	15.0	0.013	I and II	High	15.0	0.013	I-*A*, II-*A*
	0.00	High	30.0	0.016	II-*B*	High	35.0	0.018	I-*B*, II
	0.00	.....	....	.....	....	High	30.0	0.016	II-*B*
	3.00	High	17.5	0.026	I-*B*, II-*B*	High	30.0	0.009	I-*B*, II
	6.00	Low	5.0	0.067	I and II	High	35.0	0.004	I and II
	9.00	Low	5.0	0.071	I-*A*, II-*A*	Low	10.0	0.016	I-*A*, II-*A*

[1] See Fig. 33.

TABLE XI.—ESTIMATED SAVINGS IN GASOLINE CONSUMPTION IN THE FOUR GRADE REDUCTION PROBLEMS

A. Passenger cars, average gross weight 3,225 lb., assuming 1,000,000 tons per year ascending and 1,000,000 tons per year descending

Class I-*A*			Class I-*B*		
Grade			Grade		
9%	Ascending	= 32,422 gal./yr.	6%	Ascending	= 29,696 gal./yr.
	Descending	= 7,878		Descending	= 14,544
	Total	= 40,300 gal./yr.		Total	= 44,240 gal./yr.
6%	Ascending	= 29,090 gal./yr.	3%	Ascending	= 29,696 gal./yr.
	Descending	= 6,364		Descending	= 14,545
	Total	= 35,454 gal./yr.		Total	= 44,241 gal./yr.
Saving on 6% over 9%		= 4,846 gal./yr.	Saving on 3% over 6%		= 0.0 gal./yr.
Saving at $0.20/gal.		= $970 per yr.	Saving at $0.20/gal.		= $0 per yr.

B. Graham truck, gross weight 10,500 lb., assuming 1,000,000 tons per year ascending and 1,000,000 tons per year descending

Class I-*A*			Class I-*B*		
Grade			Grade		
9%	Ascending	= 29,848 gal./yr.	6%	Ascending	= 22,726 gal./yr.
	Descending	= 10,454		Descending	= 6,212
	Total	= 40,302 gal./yr.		Total	= 28,938 gal./yr.
6%	Ascending	= 27,272 gal./yr.	3%	Ascending	= 20,605 gal./yr.
	Descending	= 2,499		Descending	= 5,454
	Total	= 29,771 gal./yr.		Total	= 26,059
Saving on 6% over 9%		= 10,531 gal./yr.	Saving on 3% over 6%		= 2,879 gal./yr.
Saving at $0.20/gal.		= $2,100 per yr.	Saving at $0.20/gal.		= $575 per yr.

C. Reo moving van, gross weight 13,150 lb., assuming 1,000,000 tons per year ascending and 1,000,000 tons per year descending

Class I-*A*			Class I-*B*		
Grade			Grade		
9%	Ascending	= 23,484 gal./yr.	6%	Ascending	= 24,241 gal./yr.
	Descending	= 6,817		Descending	= 5,150
	Total	= 30,301 gal./yr.		Total	= 29,391 gal./yr.
6%	Ascending	= 30,453 gal./yr.	3%	Ascending	= 15,757 gal./yr.
	Descending	= 1,818		Descending	= 5,454
	Total	= 32,271 gal./yr.		Total	= 21,211 gal./yr.
Increased fuel on 6% over 9%		= 1,970 gal./yr.	Saving on 3% over 6%		= 8,180 gal./yr
Increased cost at $0.20/gal.		= $394 per yr.	Saving at $0.20/gal.		= $1,636 per yr.

Table XI.—Estimated Savings in Gasoline Consumption in the Four Grade Reduction Problems—Continued

A. Passenger cars, average gross weight 3,225 lb. assuming 1,000,000 tons per year ascending and 1,000,000 tons per year descending

Class II-*A*			Class II-*B*		
Grade			Grade		
9 %	Ascending	= 32,370 gal./yr.	6 %	Ascending	= 24,570 gal./yr.
	Descending	= 7,750		Descending	= 9,350
	Total	= 40,120 gal./yr.		Total	= 33,920 gal./yr.
6 %	Ascending	= 24,570 gal./yr.	3 %	Ascending	= 20,040 gal./yr.
	Descending	= 9,350		Descending	= 12,260
	Total	= 33,920 gal./yr.		Total	= 32,300 gal./yr.
Saving on 6 % over 9 % = 6,200 gal./yr.			Saving on 3 % over 6 % = 1,620 gal./yr.		
Saving at $0.20/gal. = $1,240 per yr.			Saving at $0.20/gal. = $324 per yr.		

B. Graham truck, gross weight 10,500 lb., assuming 1,000,000 tons per year ascending and 1,000,000 tons per year descending

Class II-*A*			Class II-*B*		
Grade			Grade		
9 %	Ascending	= 29,790 gal./yr.	6 %	Ascending	= 20,610 gal./yr.
	Descending	= 10,490		Descending	= 4,670
	Total	= 40,280 gal./yr.		Total	= 25,280 gal./yr.
6 %	Ascending	= 20,610 gal./yr.	3 %	Ascending	= 13,400 gal./yr.
	Descending	= 4,670		Descending	= 5,770
	Total	= 25,280 gal./yr.		Total	= 19,170 gal./yr.
Saving on 6 % over 9 % = 15,000 gal./yr.			Saving on 3 % over 6 % = 6,110 gal./yr.		
Saving at $0.20/gal. = $3,000 per yr.			Saving at $0.20/gal. = $1,222 per yr.		

C. Reo moving van, gross weight 13,150 lb., assuming 1,000,000 tons per year ascending and 1,000,000 tons per year descending

Class II-*A*			Class II-*B*		
Grade			Grade		
9 %	Ascending	= 23,340 gal./yr.	6 %	Ascending	= 22,410 gal./yr.
	Descending	= 6,700		Descending	= 4,000
	Total	= 30,040 gal./yr.		Total	= 26,410 gal./yr.
6 %	Ascending	= 22,410 gal./yr.	3 %	Ascending	= 10,150 gal./yr.
	Descending	= 4,000		Descending	= 5,170
	Total	= 26,410 gal./yr.		Total	= 15,320 gal./yr.
Saving on 6 % over 9 % = 3,630 gal./yr.			Saving on 3 % over 6 % = 11,090 gal./yr.		
Saving at $0.20/gal. = $726 per yr.			Saving at $0.20/gal. = $2,218 per yr.		

it is necessary to use certain quantities that are dependent upon the mechanical design of the vehicle and other factors determined from measurements that have been made in the course of various investigations of highway grades. None of these is established with finality as yet, but those used herein are the best available at the present time. A highway department can determine those constants it needs by a small amount of simple field work with typical vehicles.

TABLE XII.—SUMMARY OF ESTIMATED TIME SAVING BY GRADE REDUCTION

Truck	Class of grade	Grade reduction, per cent	Time saved, ton-minutes per year
Graham, gross load (10,500 lb.)........	I	9–6	13,130,000
	I	6–3	1,420,000
	II	9–6	13,130,000
	II	6–3	2,100,000
Reo van, gross load (13,150 lb.)........	I	9–6	6,900,000
	I	6–3	16,830,000
	II	9–6	10,850,000
	II	6–3	18,280,000

Tractive Resistance.—The term "tractive resistance" is usually interpreted to mean a hypothetical force whose line of action is parallel to the road surface and the longitudinal axis of the vehicle and whose magnitude is equal to the summation of the components in that line of all external forces acting upon the vehicle when it is traveling on a level surface. It may simplify the discussion of tractive resistance if its several components are considered separately.

1. *Rolling Resistance.*—When a wheel rolls over a road surface there is distortion of that surface. If the surface is of a yielding or plastic substance, a rut is formed. If it is elastic, it deflects slightly under the load but subsequently resumes its original position. In the former case considerable road surface resistance is encountered; in the latter, very little.

In either case the tire of the vehicle is distorted in the region of contact with the road surface, and some power loss results. This is dissipated in the form of heat and mechanical deterioration of the tire.

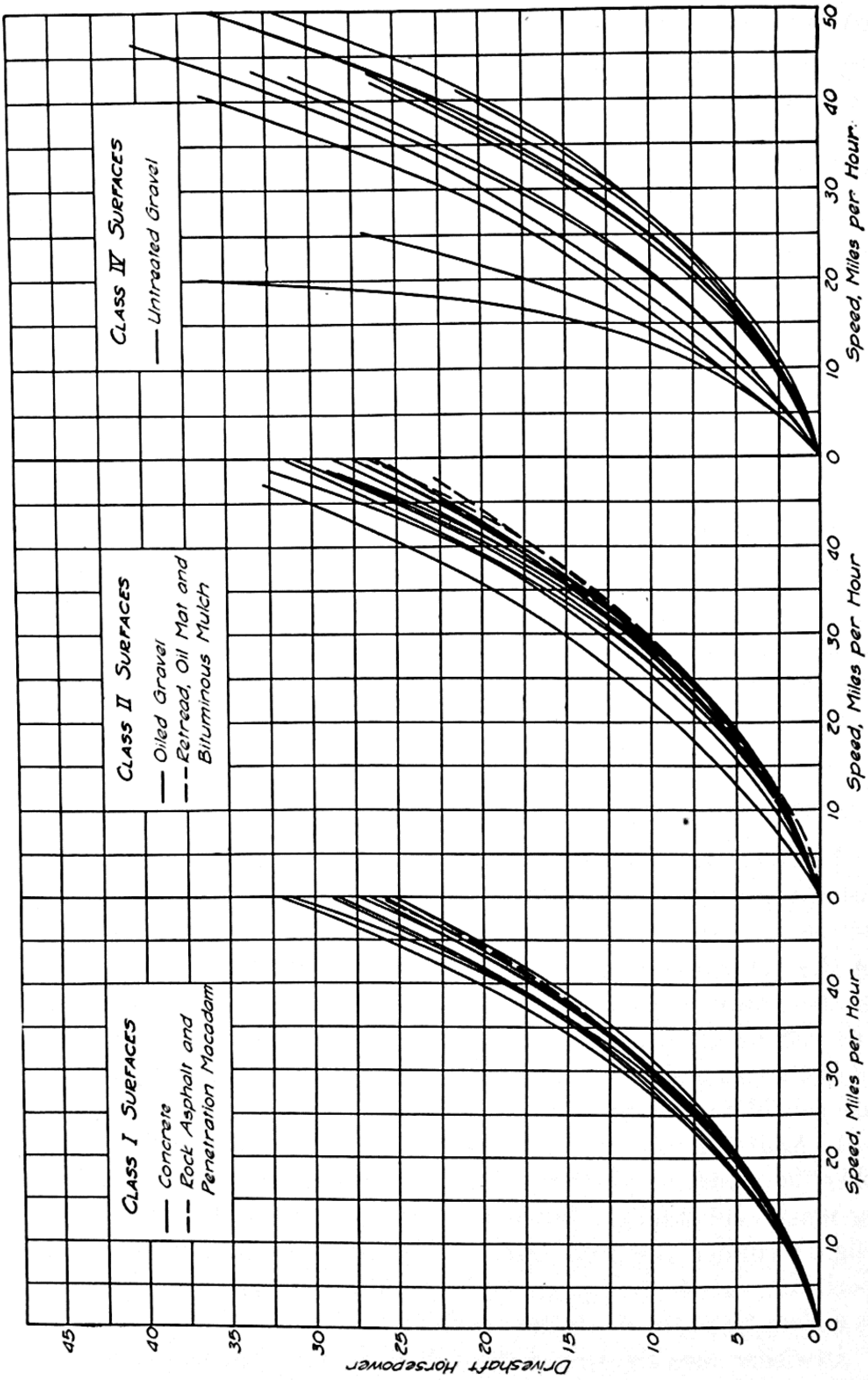

FIG. 34.—Showing typical tractive-resistance curves for automobiles.

The foregoing constitute the factors of rolling resistance, but neither varies according to a simple law. The minimum rolling resistance is a value equivalent to the power loss in the tire. For pneumatic tires this is from 22 to 28 lb. per ton of weight on the tire.[1]

The rolling resistance of a rubber tire that is caused to roll on a smooth dynamometer drum or on a circular track of any practicable diameter is a fairly definite quantity which can be measured quite accurately by determining the power loss in the tire for a period of time during which all conditions surrounding the test are held constant.

When the tire is mounted on the wheel of a self-propelled type of vehicle and drawn over a road surface or is caused to coast over that surface, the resistance to the motion of the vehicle consists not only of the equivalent of the power loss in the tires but also of the effects of air and wind, wheel-bearing friction, impact, and distortion of the road surface. The power loss in the tire is reasonably uniform under a fixed set of conditions, but when a vehicle is moving along a highway the conditions that influence rolling resistance are continually changing.

The curves in Fig. 34 show the magnitude of rolling plus air resistance for a typical automobile on several kinds of roads. Since no two roads of any type are exactly alike, and since the ratio of air resistance to weight varies for vehicles of different types and weights, it naturally follows that the resistance-speed curves obtained with a variety of vehicles on any one type of road will seldom coincide. On the contrary they are likely to form a band, the width of which reflects the effects of several variables enumerated above.[2]

2. *Air Resistance.*—The retarding effect of the air upon self-propelled vehicles is generally recognized and comprises a considerable part of the total tractive resistance. Air resistance may be computed from the formula

$$R_a = CAV^2,$$

where

R_a = the total air resistance, in pounds.

A = the projected cross-section of the vehicle, in square feet.

[1] HOLT, W. L., and P. L. WORMELEY, "Dynamometer Tests of Automobile Tires," *Tech. Paper* 240, U.S. Bur. Standards, p. 567, Sept. 24, 1923.

[2] PAUSTIAN, R. G., "Tractive Resistance as Related to Roadway Surfaces," *Bull.* 119, *Iowa Engr. Exp. Sta.*, Vol. 33, No. 9, Aug. 1, 1934.

V = the speed of the vehicle relative to the air, in miles per hour.

C = an experimental coefficient.

A value of 0.0025 for the constant C was established by extensive wind-tunnel researches[1] under the direction of Prof. L. E. Conrad at the Kansas State Agricultural College. Some later investigations[2] at Yale University indicate that the constant C may be expected to vary between 0.0019 and 0.0025 for automobiles of recent design.

Tractive resistance may be considered to consist of two groups of resistance, thus:

$$R_t = R_r + R_a, \tag{16}$$

where

R_t = total tractive resistance, in pounds per ton.

R_r = rolling resistance, in pounds per ton.

R_a = air resistance, in pounds per ton.

R_r may be expected to vary from about 22 to about 45 lb. per ton of weight of vehicle for roads of all types in good condition. It will reach 100 lb. per ton and more on mud roads and snow.

Fuel Consumption Data.—In the illustrative problem on economics of grades there were introduced certain factors having to do with speed and fuel consumption on various grades with various vehicles. Such data cannot be generalized but, on the contrary, must be worked out in each area on the basis of the prevailing types of vehicles that comprise the traffic and the types of road surfaces that predominate. The necessary data can, however, be secured in a relatively short time by using the right kind of vehicles (which probably are already owned by the highway department) and a few simple instruments.

Grades for Animal-drawn Vehicles.—In populous districts in the United States, animal-drawn vehicles constitute a negligible part of the traffic on rural highways, but in many other parts of the world reached by this treatise that class of traffic must be considered in designing highway grades.

[1] CONRAD, L. E., "Wind Resistance of Motor Vehicles," *Public Roads*, Vol. 6, No. 9, pp. 203–206, November, 1925.

[2] LOCKWOOD, E. H., "Air Resistance of Automobiles," a paper presented at the Northeast Section S.A.E., New Haven, Conn., Nov. 13, 1928, and included in the *Proc. 8th Annual Conference*, Highway Research Board, Washington, D.C., 1928.

Let

P_1 = the number of pounds pull that an animal can exert steadily for several hours, if the load is momentarily relieved at irregular intervals.

P_2 = the number of pounds pull that an animal can exert for short intervals of time.

n = the number of animals used to draw a load.

R = the rolling resistance for the vehicle and road surface under consideration, per ton of gross weight of vehicle.

T = the normal load in tons for 0.0 grades.

G = the maximum permissible grade, in per cent.

Then

$$T = \frac{nP_1}{R},$$

and

$$G = \frac{n(P_2 - P_1)}{20T}. \tag{17}$$

If conditions surrounding the project permit an exact determination of load and the weight of animals, then a more exact

TABLE XIII.—AVERAGE TRACTIVE RESISTANCE OF ROADWAYS

Applicable only to the average animal-drawn vehicle

Surface	Tractive Force Pounds per Ton
Earth, packed and dry	100
Earth, dusty	106
Earth, muddy	190
Sand, loose	320
Gravel, good	51
Gravel, loose	147
Cinders, well packed	92
Oiled road, dry	61
Oiled road, wet	108
Macadam, very good	38
Macadam, average	46
Sheet asphalt	38
Asphaltic concrete	40
Vitrified brick, new	56
Wood block, good	33
Wood block, poor	42
Cobblestone	54
Granite tramway	27
Asphalt block	52
Granite block	47

formula for maximum grade will take account of the work required to lift the weight of the animals, as follows:

$$G = \frac{n(P_2 - P_1)}{20(T + W)}, \tag{18}$$

where W is the total weight, in tons of the animals used to draw the load.

If the grade is more than 500 ft. long, it will be necessary to rest the animals about every 100 ft. beyond the first 500, when the maximum rate of grade is used.

For most draft animals P_1 should not exceed about one-tenth, and P_2 should not exceed one-third, the weight of the animal, except that for distances not greater than 100 ft. it may be taken at one-half the weight of the animal.

The values of R applicable to vehicles with steel tires may be taken from Table XIII. This table is made up of values selected from various reports of investigations of the subject and includes only dependable average values.

CHAPTER VI

THE DESIGN OF RURAL HIGHWAYS

Highway design has not been reduced to an exact science, nor can it be said, in general, that any particular design is the only one suited to a specific location. Differences in minor details and minor differences in the essential features characterize the designs developed by competent engineering organizations. There are certain fundamental principles that are quite generally recognized as being the basis of good design, but considerable latitude is allowable in adapting the design to the particular situation (which may be topographical, financial, or political) as long as the design does no violence to those basic principles. Highway design offers an excellent opportunity for the exercise of originality and good engineering judgment in incorporating recognized principles into a plan that does not necessarily follow stereotyped standards.

Width of Right-of-way.—When the design for the cross-section has been completed, the width of right-of-way required for present purposes can be determined because at that stage of the work the width required for the traveled way and the prevalence of cuts and fills will have been determined. For trunk-line highways in populous areas[1] there should be ample provision for future widening, although it is often difficult to predict what may be required. In certain areas in England steps have been taken to increase the width of right-of-way from about 85 to 125 ft., to provide for future needs. Numerous projects completed or planned in the vicinity of each of a number of metropolitan centers in the United States have required rights-of-way in excess of 100 ft. wide with considerable additional width needed in the vicinity of intersections of trunk-line highways where grade separations were planned. Although the need for such extreme widths of right-of-way arises only where multiple-lane highways are to be constructed, the cost of securing the extra right-of-way increases year by year and eventually

[1] Smith, Leroy C., "Trunk-line Highways in Metropolitan Areas," *Proc. A.S.C.E.*, Vol. 64, No. 6, p. 1138, June, 1938.

becomes almost prohibitive because of the erection of business buildings adjacent to arterial highways. This contingency has already been encountered in connection with ambitious projects in a number of states. It is particularly advantageous to secure a right-of-way of the ultimate required width whenever land is being purchased for the relocation of a highway. In such cases, the cost of the land does not vary greatly with the width of right-of-way sought (page 12).

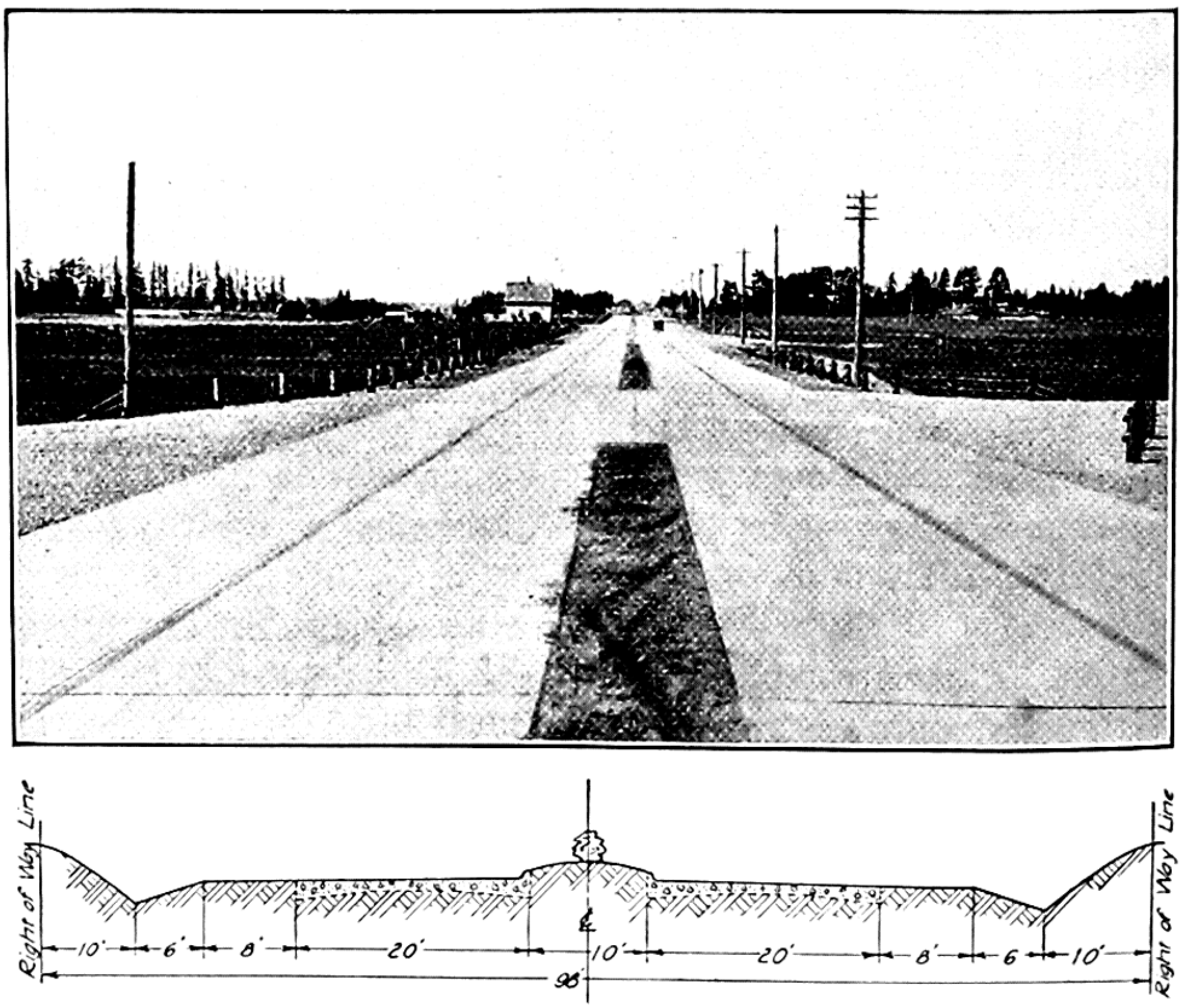

FIG. 35.—Cross-section and view of modern four-lane highway.

It has been suggested that it is sometimes feasible to secure options on the additional right-of-way that may be needed in the future, allowing the present owner to continue to use the land until it is needed for highway purposes. Meanwhile no structures would be permitted to encroach on the ultimate right-of-way.

One problem of design is to determine the width needed for present purposes and to estimate the eventual developments in so far as it is possible to do so. The requirements of the present can be determined as soon as the design for the cross-section has been completed. The cross-section of a four-lane highway and

a view of a completed road are shown in Fig. 35 to illustrate the width of right-of-way required for heavy-duty roads.

Elements of Highway Design.—The principles of design are most readily presented by considering in turn the fundamental considerations upon which the design of each element of a highway is based. The elements of highway design are:

1. The cross-section.
2. The alignment.
3. The profile, or grade line (Chap. V, in part).
4. The drainage system (Chap. III).
5. The system of signs and markers.
6. Safety features, such as guard fence.
7. Accessories.

THE CROSS-SECTION

The cross-section of a highway is shown in Fig. 36. Certain parts of this typical section may be omitted in some designs, but it shows all the elements that should be considered when developing the design. If any are omitted, it will be after due consideration of the need for that element on a particular highway. Obviously, supplementary drainage ditches and guard fences are required only at certain places; likewise, tile drains and storm-water inlets. The cross-section of a highway consists of several independent units, the design of each being developed in harmony with the other parts but on the basis of the particular function of the part. A tabulation of the basic requirements for the design of highways is given in Table XIV.

Width of Roadway Surface.—The portion of the road intended for the use of vehicular traffic is called the "roadway," and for important roads a portion of it will be provided with a heavy-duty wearing surface of some type, whereas on the light traffic roads the natural soil or some low-cost surfacing will be used. The width of the surfaced part of the roadway is fixed by the amount of traffic it is necessary to accommodate. It is considered that for each traffic lane there should be a width of 10 to 12 ft. This, however, assumes that the traffic consists principally of self-propelled vehicles, which is correct in most areas of the United States where upward of 98 per cent of the traffic is of this class. If, however, the predominating traffic is animal-drawn, as in some parts of the world, then the width of a traffic lane would ordinarily be less than 10 ft.—probably about 8 ft.

The width of roadway is, in general, that which will provide for two lanes of traffic, or, for motor traffic, say 22 ft. But if embankments were only 22 ft. wide, traffic would have to use the roadway for the full width of the embankment, which is not possible. The edges of the roadway on embankments are likely to be somewhat irregular and unstable as a result of settlement and erosion and, when wet, are seldom firm enough to carry traffic. A greater width of roadway must be provided on embankments than on sections in cut or where the normal side ditch is provided. There is no standard for this extra width, but experience indicates that it should be not less than 4 ft. along each edge. Where two lanes only are to be provided for unsur-

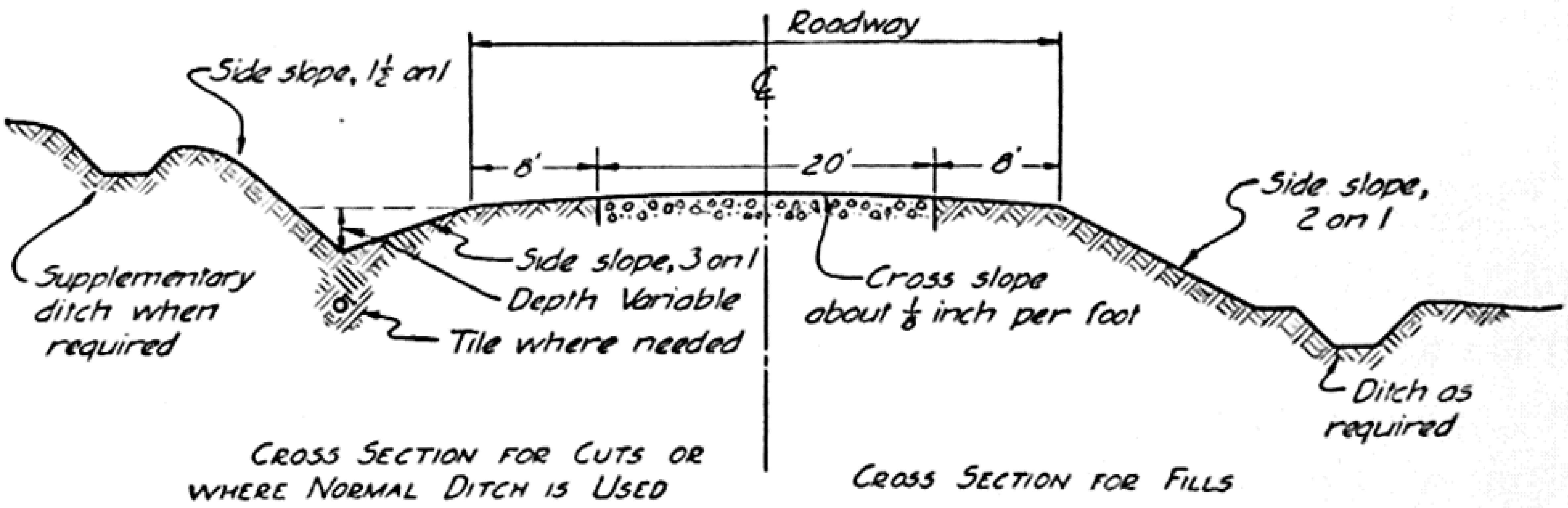

FIG. 36.—Cross-section showing the elements to be considered in design.

faced roads or those with the light traffic types of surfacing, the minimum width of roadway should be 24 ft. in a normal cross-section with the usual side ditches, and 32 ft. on embankments. Such a design would be suitable for roads that carry a moderate volume of traffic, perhaps up to 150 vehicles per day. Vehicles would not use the outer 5 or 6 ft. of the roadway except when passing other vehicles, which would not occur frequently on a road of this class.

When the traffic is sufficient in volume to require a heavy-duty type of wearing surface two lanes wide, there is passing traffic to consider on only one side of the lane (the left-hand lane in the United States), and it is not always necessary to provide a width of 12 ft. per lane for the kind of traffic to be accommodated, but there are few who consider less than 10 ft. per lane as adequate. Consequently the standard two-lane wearing surface is generally not less than 20 ft. wide when automobile traffic predominates or 24 ft. wide where the traffic includes a considerable percentage of commercial vehicles. Where the highway carries a high percentage of truck and bus traffic, the width should be at least 24 ft.

The designer must take into account two other factors that are injected into the problem when a wearing surface is to be constructed on the roadway. The first of these is the necessity for providing enough width of berm alongside the wearing surface to make certain that the edge of the wearing surface is adequately supported. The second is that vehicles will sometimes be compelled to stop along the road for various reasons, and there should be a berm alongside the wearing surface of sufficient width to permit such vehicles to draw out of the traffic lane, which requires a berm width of at least 8 ft. Preferably the berm should be 10 ft. wide.

If the traffic volume requires more than two lanes, three lanes may be the most economical, although not the most desirable, solution. The three lanes would be provided by a roadway surface of the required width, marked for three lanes. If four or more lanes are required, the right-hand lanes, two or three in number, are separated from the left-hand lanes by a parkway at least 10 ft. wide. It is the intention to make it impossible for vehicles traveling on the right-hand lanes to encroach on the left-hand lanes. Traffic connection between the right-hand and left-hand lanes will be at infrequent intervals, such as main highway intersections.

Traffic Capacity.—In many cases it is unnecessary to analyze the traffic on a highway before undertaking the design. The usual experience is that the first improvement will be a two-lane wearing surface, the exact width of each traffic lane being determined by the designer's knowledge of the general character of the traffic and the certainty that the volume of traffic does not exceed the capacity of a two-lane road.

The development of relief roads in areas of congested traffic, the fixing of highway bridge roadway widths, and the design of express highways for long-distance fast traffic in areas capable of furnishing great volumes of traffic, all involve estimates of the volume of traffic that must be accommodated and the traffic capacity of a proposed design under the particular type of traffic control that will probably exist.[1]

The maximum traffic capacity that can be secured under practical operating conditions is obtained when the vehicles are segregated into lanes and required to travel at a predetermined

[1] McIntyre, Lewis W., "Causes of Failure in Handling Traffic," *Proc. A.S.C.E.*, Vol. 63, No. 9, p. 1742, November, 1937.

speed and are not permitted to pass the vehicle ahead. This condition is secured only under very rigid traffic regulations and adequate enforcement. The Holland tunnel under the Hudson River is operated under these conditions and accommodates a maximum of about 1,900 vehicles per hour in one lane, the

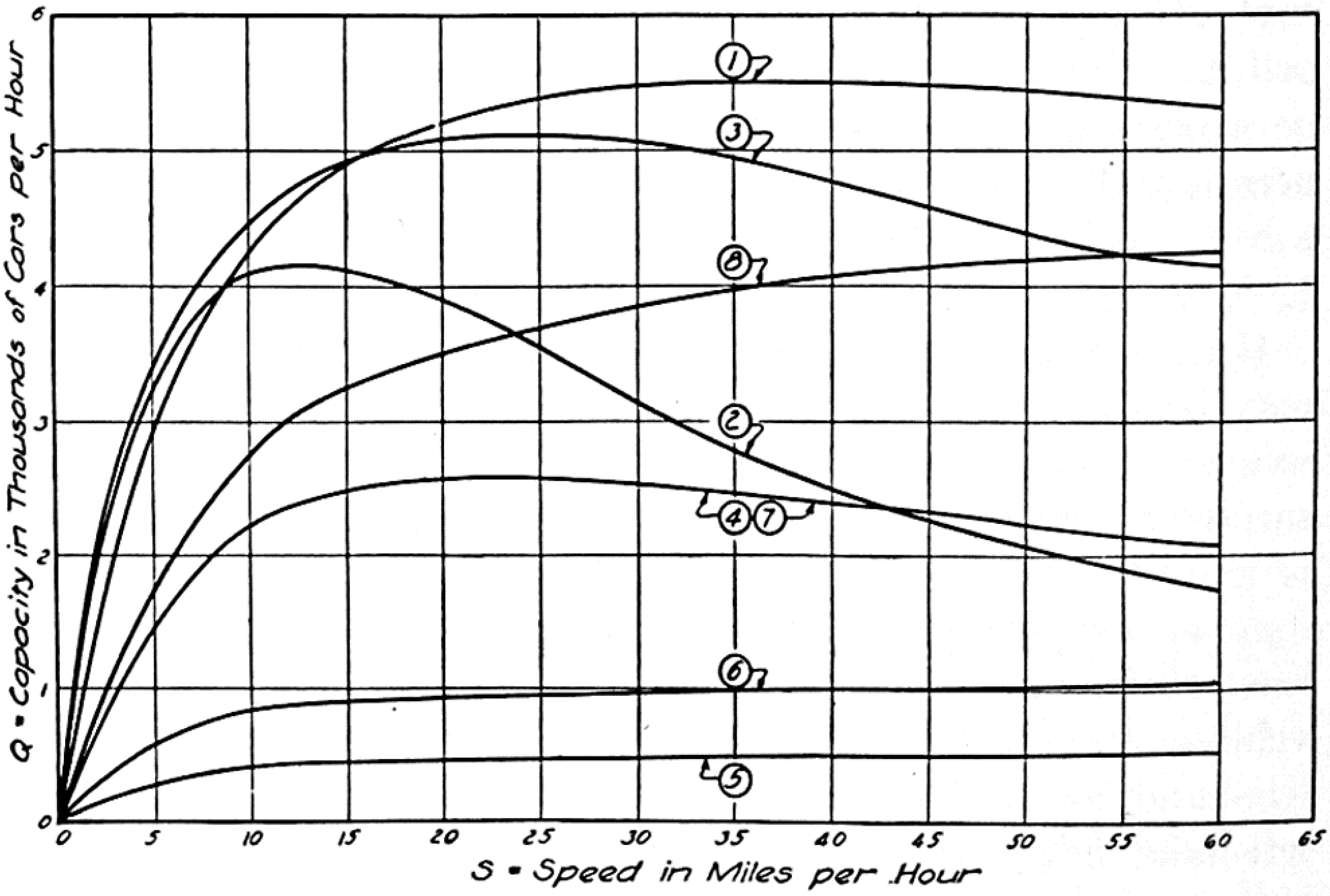

FIG. 37.—Traffic capacities under specified conditions.

Curve 1. Based on Johnson's formula, $Q = \dfrac{(S)(2)(5280)}{(16) + (0.5S^{1.3})}$.

Curve 2. Based on author's formula, $Q = \dfrac{(S)(2)(5280)}{16 + 0.095(S)^2}$.

Curve 3. Based on Greenshields' reaction-time and safe-stopping distance formula, $Q = \dfrac{(S)(2)(5280)}{16 + [(S)(1.47)(1.5)]}$.

Curve 4. Based on Moyer's reaction-time plus safe-braking distance formula,

$$Q = \frac{(S)(2)(5280)}{16 + (1.47S)(1.5) + \dfrac{S^2}{(30)\ (0.6)} + 5}.$$

Curve 5. Based on rigid control that will permit free passing conditions on a two-lane road.

Curve 6. As for curve (5), except for a three-lane road.

Curve 7. For a four-lane road upon which traffic is able to travel freely at the speed indicated by the abscissae.

Curve 8. For a four-lane road upon which traffic moves at the uniform speed indicated by the abscissae.

vehicles traveling at about 35 miles per hour. Similar conditions exist on certain highways in or adjacent to large cities that are operated as through routes restricted to passenger traffic. The theoretical traffic capacity of two- and four-lane highways under this operating condition is shown by curves 4 and 8 in Fig. 37.

These capacities are approached quite closely, as evidenced by the fragmentary traffic counts on several busy highways given in Table XV, page 168.

The traffic capacity of a highway under congested conditions[1] may be considerably higher than under the conditions outlined in the preceding paragraph, but the speed will be greatly reduced, and the driving conditions very unsatisfactory to the road users. When the congestion develops only under conditions that recur infrequently (holidays, sports occasions, and the like), it may be a wise policy to accept the delay rather than expend the money required to improve the situation as long as it is possible for the traffic to move. The traffic that may be passed over a road under these conditions is variously estimated by means of traffic capacity formulas developed on the basis of the analysis of traffic records and certain assumptions as to the behavior of traffic. Curves 1, 2, and 3 of Fig. 37 show the results of the application of three formulas for estimating traffic capacity. It should be noted in particular that curve 2 reflects the effect of speed on traffic capacity due to the "bunching" of vehicles that takes place under these conditions. Curves 1 and 3 are based on the assumption that nearly all the "bunching" can be eliminated by suitable patroling.

The most desirable condition to be established in providing traffic capacity is the one in which each vehicle is able to travel at the desired speed, as long as within the maximum permitted by law, without being appreciably delayed in passing vehicles that are traveling more slowly. Of course, a condition cannot be established where there is no delay whatever in passing, but curve 5, Fig. 37, shows the traffic capacity when that condition is approached. If the traffic on a two-lane road is of a mixed character and averages more than 500 vehicles per hour, there will be periods of congestion and delay; any improvement undertaken on such a road should, among other things, provide additional traffic lanes. If it is a three-lane road, the traffic capacity will be about 1,000 per hour without congestion, and a four-lane road will carry about 2,500 per hour without congestion. These capacities are shown by curves 5, 6, and 7 of Fig. 37.

The significance of traffic volume in highway design may be summarized thus:

[1] Greenshields, Bruce D., "Studying Capacity by New Methods," *Civil Eng.*, Vol. 5, No. 5, p. 301, May, 1935.

1. Congestion begins to be noticeable on two-lane roads when the total traffic reaches 500 to 600 vehicles per hour, and on a four-lane road at a discharge of about 2,500 vehicles per hour.

2. A two-lane road under conditions of congestion may discharge as many as 2,000 vehicles per lane per hour if traffic is held in the lane by traffic patrols, and a four-lane road up to 5,000 vehicles per hour, but the average speed will be much less than when the traffic is within the normal capacity of the road.

3. Formulas for estimating traffic capacity[1] are useful for approximate forecasts only, but traffic flow records for highways of a similar character must be studied and evaluated before final determination of the design required for a specific project. Several formulas for estimating traffic capacity are given in connection with Fig. 37.

4. The alignment of the road, the sight distance, the gradients, the effectiveness of traffic control, and the weather, all affect the traffic capacity of any road.

Drainage Ditches.—The cross-section usually adopted for highways naturally lends itself to placing the ditches at the side of the traveled way. The ditch is always an element of danger and especially so when considerable capacity must be provided and the transition from the traveled way to the bottom of the ditch should not be too abrupt. A slope of 3 on 1 is about right, and for a ditch 2 ft. deep there will be required a width of 6 ft., plus the back slope width. The back slope of the ditch is usually made as steep as the natural soil will stand, which is about 1½ on 1. The width required for back slope will be variable, depending upon the depth of cut.

For fills, the side slope will be what the natural soil will stand when finally compacted, which is about 1½ on 1 for most soils. The practice of compacting the fills as they are built up is becoming standard, and they can be built with any desired side slope. The test of stability comes after a period of several years of weathering. Along the fills the side drainage ditch will be on the low level, out a few feet from the toe of the slope.

The use of tile has been discussed in the chapter on drainage and need not be taken up here except to call attention to the fact that the layout of the tile system is a part of the problem of design in those instances where tile is needed. Express highways with four or more lanes separated by a narrow parkway introduce new problems of highway drainage which can be

[1] SHELTON, W. ARTHUR, "Methods of Estimating Highway Traffic Volume," *Proc. Highway Research Board*, Vol. 16, p. 239, 1936.

solved only by the extensive use of tile drains with inlets, much after the style used in street drainage.

Cross-slope.—The roadway must be provided with cross-slope for drainage, except in arid regions. In the normal design, drainage ditches are provided along each side of the roadway, which is constructed with a convex surface and is said to be "crowned."

The difference in elevation between the edge of the roadway and the highest part thereof is the "total crown." The total crown divided by the distance in feet from the edge to the highest part is the "rate of crown." The longitudinal line at the highest part of the roadway is the "crown line" and in normal cross-sections is at the middle of the roadway.

The rate of crown is fixed as a compromise between two conflicting requirements. The greater the cross-slope the better the drainage, and it is very desirable that the storm water run off the roadway quickly, especially when the wearing surface is of some absorbent material such as earth, gravel, or sand-clay. But the greater the cross-slope the more the tendency for vehicles to slide sidewise when the surface is slippery. To compromise these divergent considerations with reference to cross-slope, it is customary to hold the rate of crown to the minimum consistent with acceptable drainage of the wearing surface. The usual maximum rate of crown is about ½ in. per foot for fairly dense natural soils, gravel, and sand-clay types and ¼ in. for impervious surfaces. A good working rule for all non-absorbent surfaces is that the total crown should be not more than one-seventy-fifth and not less than one one-hundredth of the width, when the crown line is in the middle. The equivalent rates of cross-slope are to be followed for unsymmetrical cross-sections.

ALIGNMENT

The general route of a road is fixed by the principles governing the establishment of the particular system of highways of which it is a part. The exact location of the sections of the highway between towns is a problem of engineering economics that is solved by an analysis of location inspections and surveys which are carried out as preliminaries to construction. As far as possible, an existing right-of-way is utilized, but in many instances minor and even major relocations of the route will be necessary to secure correct alignment.

Four main considerations enter into the establishment of the final location: (1) directness, (2) grades, (3) safety, (4) esthetics.

Directness.—The location of a road is based on the desire to secure a connection between towns or districts. Generally there is no reason for improving any other than the shortest possible route. In studying the location, the question of shortening the existing route by means of relocations will present itself. When such proposed relocations do not involve questions of safety or grades, the justifiable expenditure for the relocation will be based on time- and cost-saving to vehicle operators. The economic factors to be considered in planning relocations to save distance are discussed on page 434.

Grades.—Two methods are available for reducing the rate of grade on a highway. One is to reduce the height of the hill by excavating at the top, filling at the bottom, or both; the other is to increase the distance traveled in ascending a hill, thereby reducing the rate of grade. This latter method is employed in very hilly or mountainous country, and the best alignment possible is selected in view of the topographical limitations. The location is certain to involve a great deal of curvature. The distance between terminals will generally be increased by relocations of this type, and there is no way to avoid the increase.

When short relocations are being considered with a view to avoiding unfavorable grades, the fact that increased distance means an increase in cost of transportation must be taken into account. The saving brought about by reducing grades is to be balanced against the increased cost of distance, and the decision as to the alignment to adopt will be determined by the economic facts thus deduced (Chap. V).

Safety.—Curves of short radius, restricted sight distance, high embankments of inadequate width, locations subject to landslips, grade crossings with other transportation lines, obscure intersections with other highways, and similar physical and topographical features of a route contribute to accidents. Adjustments of alignment in the interest of greater safety are justifiable regardless of the effect upon distance. Many relocations are necessary to secure safety, and these are adopted without regard to the economic aspects of the problem.

Esthetics.—Wherever it is possible to locate a road so that travelers may enjoy beautiful scenery, it is well worth the time and effort required. Sometimes major relocations are under-

taken in order to traverse regions of natural beauty. With a little care in location, beautiful vistas, imposing bridges, villages nestling in a wooded valley, and similar inspiring scenery can be brought into the range of vision of those who traverse a highway.

HORIZONTAL CURVES

The ideal highway would be one laid out on a tangent between termini, but of course there are few areas where location is such a simple problem. Where there is a change in alignment, a curve is used to connect the two tangents. In extreme cases the alignment may be a succession of curves.

Rates of Curvature.—The curve constitutes an unavoidable element of highway location but is an accident hazard unless it is so easy that vehicles can safely travel on the curve at the same speed as on the tangents. This is possible on curves of about 2° 30′ (radius 2,292 ft.), and the aim should be to use such curves on the heavy traffic routes (1,000 or more vehicles per hour) whenever topography permits. At least it will generally be feasible in most instances to hold the curves to 5° 0′ (radius 1,146 ft.) or flatter on routes of this character. On routes of somewhat lighter traffic, say below 500 per hour as a maximum, the curves may be somewhat sharper but should not be sharper than 8° 0′ (radius 716 ft.). On the minor trunk lines, county roads, and local feeder roads the 8° 0′ maximum should be attempted, but the volume of traffic will not justify the increased cost of the flat curves where the topography is unfavorable.

When a vehicle passes from a tangent to a horizontal curve the driver must adjust his steering to the new path, and this requires a little time. If he passes directly on to a circular curve, he will weave about a little until his steering is adjusted; that is, he follows roughly a spiral path instead of a circular. It seems self-evident that he should be provided with a spiral path to facilitate his travel.

Easements.—The use of easement curves has become common in highway practice because it aids the traffic to hold speed in passing from tangent to curve, and for any curve where superelevation is adopted the easement spiral is necessary to a neat and harmonious development of the construction of the curve.[1] The normal crowned cross-section of surface is used for tangent

[1] PRITCHETT, C. M., "Spiral Curves Cut Costs of North Carolina Road Work," *Eng. News-Record*, Vol. 93, No. 4, p. 132, Nov. 13, 1924.

locations, but the instant the vehicle passes from the tangent to a horizontal curve it should be provided with the proper superelevation for that curve. Obviously the surface cannot conveniently provide for instantaneous change from crown to superelevation, and therefore a transition section is provided which has a normal crowned cross-section at one end and the maximum superelevation at the other and a gradual transition from one to the other. In the older designs this transition section was a short tangent to the circular curve extended to intersect the tangent stretch of road. There was nothing desirable about this system except that it was rather simple to construct. The correct method of providing the transition is by means of some form of transition curve which may be a lemniscate, a cubic parabola, or some special form of "spiral." Since, in any case, tables[1] are provided to simplify the calculations, the adaptation of these curves to a specific design is not difficult.

The length of the spiral should be great enough to permit reaching maximum superelevation without an abrupt change in the slope of the surface and to give the driver sufficient time to turn his steering wheel enough to provide the "slip angle" needed to develop the friction required to change the motion of the vehicle from a lineal to a curved path[2] and time to readjust the steering wheel to the position required on the curve. The transition curve, in brief, should be long enough to enable the vehicle to change its path without hazard or discomfort to the occupants.

A vehicle traveling at a constant speed on a horizontal curve is accelerating toward the center of curvature ("normal" acceleration) at the rate of V^2/R, in which V represents the speed in feet per second and R represents the radius of the curve in feet. If an easement curve is employed, the radial acceleration on it will vary from zero at the T.S. where R = infinity, to V^2/R at the S.C. The time in which this acceleration is attained on a spiral of length L_s is L_s/V, which should be the time required to change the path of the vehicle travel safely and comfortably. The rate at which the vehicle changes its direction (normal acceleration) while moving along the spiral is equal to the normal acceleration

[1] For example, Thomas F. Hickerson, "Highway Curves and Earthwork," McGraw-Hill Book Company, Inc., New York, N.Y., 1926.

[2] Moyer, R. A., "Skidding Characteristics of Automobile Tires," *Bull.* 120, *Iowa Engr. Exp. Sta.*, p. 92, August, 1934.

divided by the time, or

$$\text{Rate of change of normal acceleration} = C = \frac{V^2/R}{L_s/V} = \frac{V^3}{L_sR}. \quad (1)$$

Conclusive data as to the appropriate value for C have not been obtained. In England $C = 1$ is favored. In the United States certain state highway departments are using a value of $C = 3$ in planning highways for a speed of 100 m.p.h. The driver of a fast-moving vehicle must have time and space to bring his steering wheel to the position required by the degree of curve plus the additional amount required to develop the necessary friction between road and front wheels to hold the car on the curve.

On the basis of road tests and other observations at Ames,[1] a length of transition spiral based on a $C = 2$ ft. per second per second was found to be satisfactory. Racing drivers at Indianapolis, Ind., and Des Moines, Iowa, developed a speed on curves that indicated that a value for C as high as 5 to 6 ft. per second per second is possible. It seems reasonable to adopt $C = 2$ for ordinary highway traffic, which allows a factor of 3 to compensate for the ineptitude of the run-of-the-mill drivers.

The formula for the length of transition is $L_s = V^3/CR$, where

L = the length of the transition section, in feet.
V = speed, in feet per second.
R = radius of curvature in feet.
C = a constant representing rate of normal acceleration.
S = the speed in miles per hour.

With $C = 2$, the formula for length of spiral becomes

$$L_s = \frac{1.58S^3}{R}. \quad (2)$$

This equation is based on providing a comfortable rate of changing direction and an acceptable length of spiral for high speeds. It will be noted in Table XVI that spiral lengths are given for speeds up to 80 m.p.h., which is in harmony with the superelevations recommended in Fig. 38. There are obviously a number of combinations of lengths of spiral and degrees of curve that may be worked out for any site, and conditions at a site may require a spiral so long that the curve becomes transitional throughout its entire length. A corollary of this is that

[1] *Ibid.*, p. 113.

the curve chosen for any site should be long enough to permit laying out double the length L_s, the transition spiral that is to be used instead of the circular curve.

Theory of Superelevation.—When a vehicle is traveling in a curved path on a horizontal plane, the centrifugal force tends to cause the vehicle to slide sidewise and away from the center of curvature. The intensity of the centrifugal force may be calculated from the familiar equation

$$F_1 = \frac{MV^2}{R} = \frac{WV^2}{gR}, \tag{3}$$

where

F_1 = the centrifugal force, in pounds.
W = the weight of the vehicle, in pounds.
M = the mass of the vehicle = $W/32.2$.
V = the tangential velocity of the vehicle, in feet per second.
R = the radius of curvature, in feet.

If the vehicle is standing, or moving in a straight line, on a plane that is inclined at an angle θ with the horizontal, there is a force tending to cause it to slide or roll down the plane. The magnitude of this force for a vehicle weighing W lb. is:

$$F_2 = W \tan \theta. \tag{4}$$

Sliding, because of centrifugal force or the inclination of the plane, is opposed by the friction between the tires and the surface. The magnitude of this resistance is equal to the normal pressure between the tires and the surface, multiplied by the coefficient of friction for the tires on that surface. When dealing with self-propelled vehicles, account must be taken of the fact that the driving wheels usually slide before the others, as a result of the slippage from the driving effort.

To add to the safety in driving on horizontal curves the road surface, including the berms or shoulders, is constructed with the cross-slope rising from the inside of the curve for the entire width of the surface; that is, instead of being crowned, the traveled way is superelevated. Animal-drawn traffic may be disregarded in the design of superelevation, and the weight assumed for the vehicle may be considered instead of the normal pressure between wheels and road, since for the rates of superelevation employed in practice the two are so nearly of the same magnitude.

The theoretical superelevation required for any horizontal curve, when no account is taken of the effect of friction, may be calculated by assuming that F_1 and F_2, Equations (3) and (4), are to be equal; hence there is to be no tendency for the vehicle to slide in either direction. Then

$$\tan \theta = \frac{V^2}{gR}. \tag{5}$$

In order to reduce this relation to terms readily applied in practice, the following are introduced:

e = the superelevation, in feet per foot of width.
S = speed, in miles per hour.
g = 32.2.
R = radius of curvature to the center line of the pavement, in feet.

In these terms, the superelevation formula becomes

$$e = 0.067\frac{S^2}{R}. \tag{6}$$

The effect of friction between tires and road surfaces may be introduced as follows:

Let

p = the percentage of the weight that is carried by the rear wheels of the vehicle.
f = the coefficient of friction between the tires and the road surface.

Then

$$W \tan \theta = \frac{WV^2}{gR} - pWf,$$

and

$$e = 0.067\frac{S^2}{R} - pf. \tag{7}$$

Equation (7) is convenient for estimating the conditions that will exist on a curve that has less than the theoretical superelevation and is of low frictional resistance because of ice or some other coating on the road surface.

Applications of Superelevation.—The design of a road surface for a curve connecting two tangent sections of a highway requires that e be calculated for a stated speed and coefficient of friction

or, more properly, frictional force aiding the superelevation in preventing lateral skidding. If the vehicle travels faster than the calculated speed, a larger frictional force than assumed in the calculations must be available, or skidding will take place; similarly, if the vehicle travels more slowly than the speed assumed in the calculations, there will be a tendency to slide

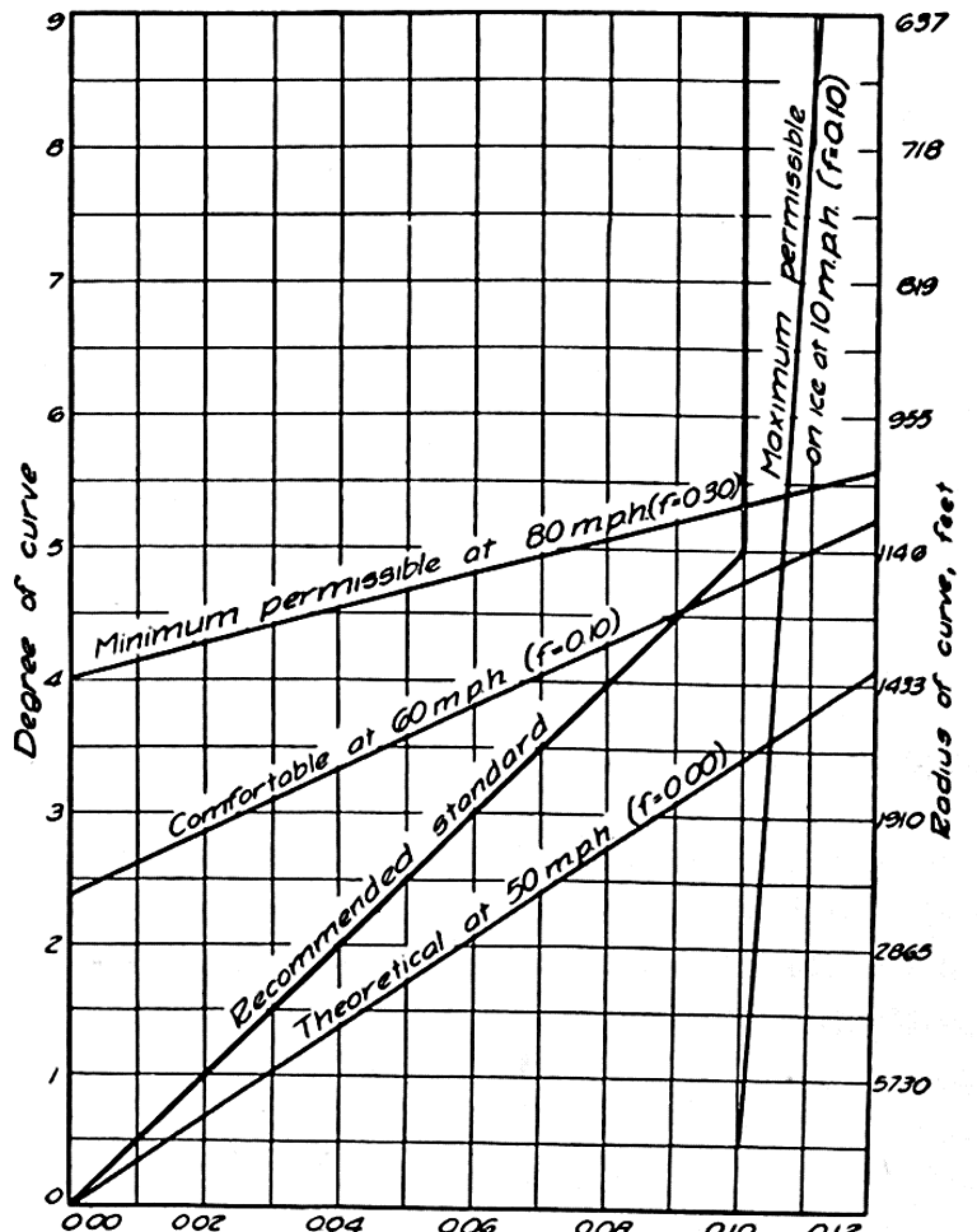

FIG. 38.—Superelevation requirements on main trunk highways.

toward the inside of the curve. Either of these conditions may cause discomfort and sometimes danger to the occupants of the vehicle, particularly when the pavement is slippery.

A recommended standard for superelevation has been developed at Iowa State College[1] in which proper weight has been given to all factors that affect the superelevation, with comfort and safety for the ordinary road user under normal driving condi-

[1] *Ibid.*, p. 86.

tions as a major consideration. The suggested *maximum* superelevation is based on a coefficient of friction ($f = 0.10$) which is about the maximum for an icy surface and a speed as low as 10 miles per hour for safe operation on such surfaces. This is applicable only in regions where snow or sleet is to be expected. The suggested *minimum* superelevation is based on a coefficient of friction ($f = 0.30$) which is about the maximum available for vehicles at normal speed on a dry surface. Coefficients of friction higher than 0.30 have been measured and could be expected under certain favorable conditions, but for general design purposes it is unwise to employ these higher exceptional values.

The rates of superelevation recommended for main trunk highways are shown in Fig. 38.

Superelevation of the road surface on the circular part of a curve is generally provided by constructing the surface to a uniform slope from edge to edge, and the cross-section is unchanged from the S.C. to the C.S. That is, the road surface is neither crowned nor dished and has the same cross-slope from beginning to end of the part that is a circular arc. Sometimes surfaces with low rates of crown are built with the crowned cross-section for the superelevated part as a construction convenience, but this is not very common practice. The superelevation on the transition section of the curve, which may be a tangent or a spiral, increases gradually from the normal crowned section at the T.S. to full superelevation at the S.C. The center-line grade may be carried around the curve, in which case the cross-section appears to have been tipped by rotating about the center line. This is probably the most common design. The grade of the inside edge of the pavement may be carried around the curve, in which case the cross-section appears to have been tipped by rotating about the inside edge.

The coefficients of friction between tires and road surfaces according to measurements made at Iowa State College are summarized in Table XVII.

Widening Wearing Surface on Curves.—Moyer[1] has shown that whether or not the wearing surface on curves is built with a spiral transition section, vehicles will be compelled to follow a spiral path on the transition section and that in so doing they will encroach on the lane adjacent to the one they are supposed

[1] *Ibid.*, p. 120.

to follow, unless the wearing surface is built with sufficient width to permit following a spiral path in each lane. If wearing surfaces on curves are not spiraled, they must be built wider than on tangent stretches by an amount that will provide room for the vehicle to make its own spiral without encroaching on the adjacent lane.

The amount of this extra width is given with sufficient accuracy by the formula

$$W_s = \frac{S^6}{40R^3}, \tag{8}$$

where

W_s = the increased width needed to permit speed on curves that are not spiraled, in feet.

S = the speed, in miles per hour.

R = the radius of curvature, in feet.

In addition it must be recognized that in rounding a curve the front wheels of the vehicle follow a path of longer radius than that of the rear wheels. This extra width of path is given with sufficient accuracy by the formula

$$W = \frac{L^2}{2R}, \tag{9}$$

where

W = the extra width, in feet.

L = the length of vehicle wheel base, in feet.

R = the radius of curvature of the path of the vehicle, in feet.

The lane on the curve must be wide enough not only to provide for the spiral path but also to accommodate the increased road width of the path of the vehicle on curves. If curves are not spiraled, the extra width required is therefore the sum of the requirements for extra tread width and that required for speed, which is

$$\frac{L^2}{2R} + \frac{S^6}{40R^3}. \tag{10}$$

It will be noted that for long radius curves—700 ft. or more—and speed of not more than 60 m.p.h. the amount of widening required is of the order of 2 or 3 ft. It will increase rapidly as the radius decreases, and the speed would have to be decreased to keep the road surface to a practical width.

Where the wearing surface on horizontal curves is constructed with a correctly spiraled easement curve with 12-ft. lanes, no

additional width is required on curves of 8 deg. or less. If the project is designed with 10-ft. lanes, the wearing surface on the curves should be widened to 12 ft. for curves of 8 deg. or less to

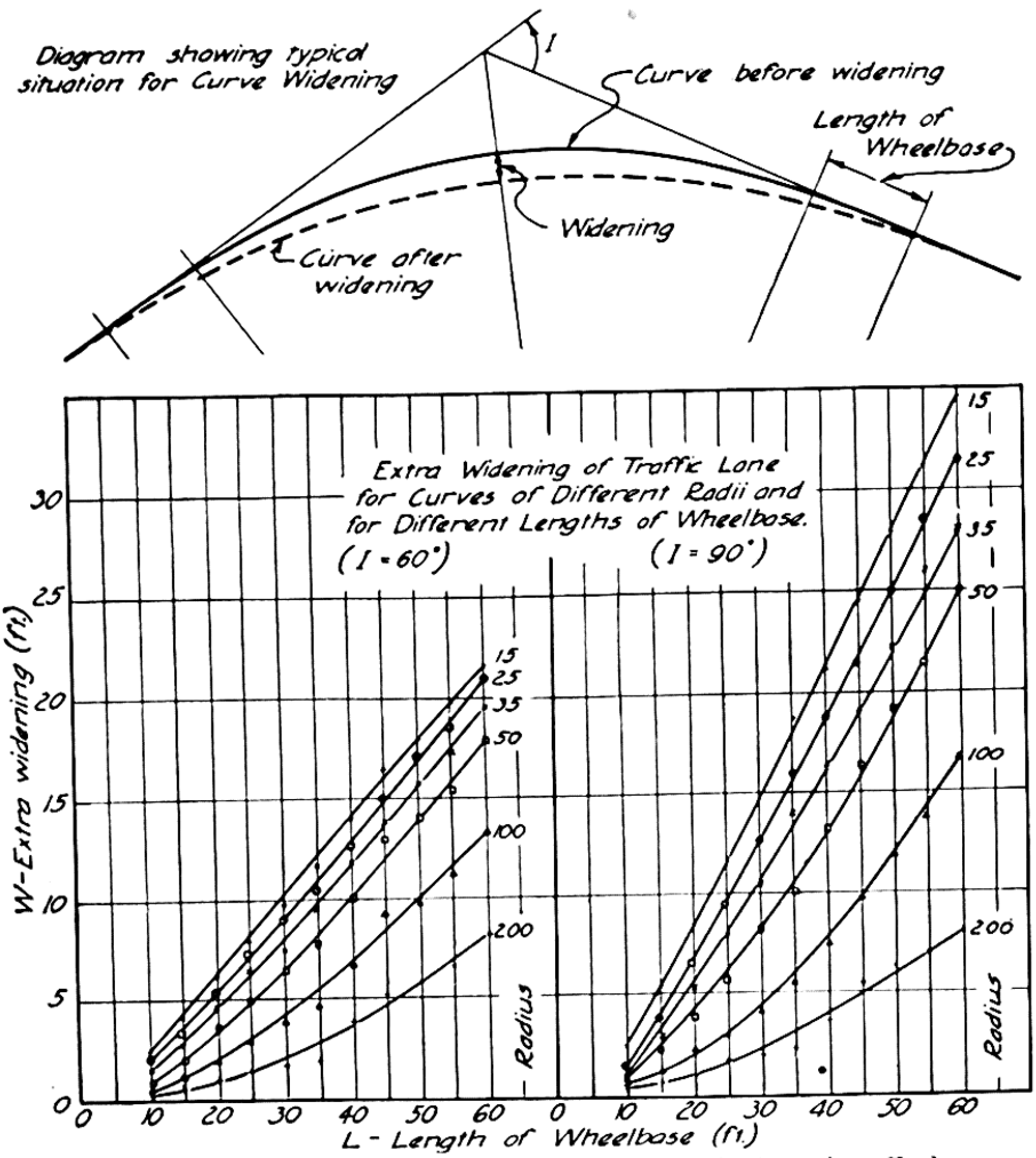

FIG. 39.—Showing the basis for calculating the widening of traffic lanes.

take care of the $L^2/2R$ factor. Where the wearing surface is constructed with spiraled curves of more than 8 deg., the wearing surface on the curves should be widened in accordance with Formula (9), the speed upon which the design is based being determined by the conditions of the location but generally being less than 50 m.p.h. (see Fig. 39).

VERTICAL CURVES

At those places where the rates of longitudinal grade change, the transition is made safe and sightly by introducing vertical

curves at the summits and sags. In planning the grades for highways in hilly regions it is often advantageous to employ vertical curves almost to the exclusion of straight-line grades because of the saving in earthwork costs that can be effected thereby. There are few basic principles involved, and it is a simple matter to adapt the vertical curve to the conditions of a specific design.[1]

Vertical Curves at Summits.—When two vehicles approach the top of a hill from opposite directions on a highway at least two lanes wide, there is no element of danger if each is held to its proper lane and the drivers are able to see each other while they are still a reasonable distance apart. The line of sight of an automobile driver is about 5 ft. above the road surface. With that factor fixed, the curvature is readily computed for any desired sight distance. The problem then becomes one of determining what constitutes a reasonable sight distance, but upon this point it is not easy to be specific. Perhaps a good basis for preliminary computations is to determine how much distance is required to bring a vehicle to a stop from the extreme road speed to be expected (if there is any such thing as a limit to speed, which seems doubtful). If the road surface permits a reasonable application of the brakes without starting a skid, a vehicle with four-wheel brakes could be stopped in about 300 ft. from a speed of 60 m.p.h. To this must be added about 75 ft. as the distance traveled during the "reaction time." This would indicate that about 800 ft. is the minimum sight distance for summits on busy trunk-line highways. Many of the state highway departments are designing the trunk highways with a sight distance of 1,000 ft. or more. On roads in the secondary system where the speed is lower it is easy to deduce that a sight distance of 350 ft. is enough.

The recommended sight distances for various classes of highways are indicated in Table XIV.

The circular arc is the correct form of curve to use for vertical curves because it insures the same sight distance whatever the relative position of the vehicles when they first sight each other. As a matter of construction convenience the parabolic form of vertical curve is often employed, and for sight distances up to 500 ft. the two curves are so much alike that it matters little

[1] FURR, M. W., "Application of Vertical Curves to Highway Design," *Eng. News-Record*, Vol. 20, No. 96, p. 819, May 20, 1926.

which one is used. With the longer vertical curves required for a sight distance of 800 ft. or more the parabolic grade line will require materially heavier grading than the circular. The extent of this can readily be checked in connection with a specific design.

Vertical Curves in Sags.—The minimum radius of vertical curvature in sags is fixed by the designer's judgment in the matter of appearance. Any curve that would be accepted from the standpoint of appearance will be of satisfactory riding quality. In most instances the vertical curves in sags are selected with a view to fitting the ground without excessive earthwork and are longer than would be required for other reasons.

Rolling Grade Lines.—In certain types of topography, notably where the highway leads over a succession of hills requiring grades upward of 500 ft. long, a grade line is established that follows the existing topography quite closely. These are sometimes referred to as "rolling grades." There is ample justification for the adoption of rolling grades in certain locations, and the established grade line will be a succession of vertical curves selected to fit the existing topography as nearly as may be, with only short sections of straight grade connecting the vertical curves, but the curve so adopted should have at least the minimum radius needed to provide ample sight distance.

Grade Separations.—The separation of the roadways at the intersection of trunk highways outside the cities and towns has become a necessary safety provision. The structures required for these separations are tremendously costly and require a large area of land if they are to be adequately handled (40 to 80 acres or more), and the design is exceedingly intricate. The principles involved, however, are few and simple. The conventional designs are all of the type that has come to be known as the "clover-leaf" intersection, and variants thereof.[1] The principles involved are:

1. Gradients and sight distances complying with the standards already set forth herein.
2. No left turns for traffic passing from one route to the other.
3. Curves of the radius and superelevation required for the speed needed to prevent congestion, which will depend upon the percentage of the traffic that changes direction at the intersection.

[1] Swan, Herbert S., "Separating Grades at Highway Intersections," *Civil Eng.*, Vol. 3, No. 2, p. 79, February, 1933.

4. Pavement widened to provide deceleration lanes for the traffic that must slow down to make a turn in leaving a route.

5. Pavement widened to provide acceleration lanes to permit traffic to speed up before feeding into the fast-moving stream on the route to which the change is being made.

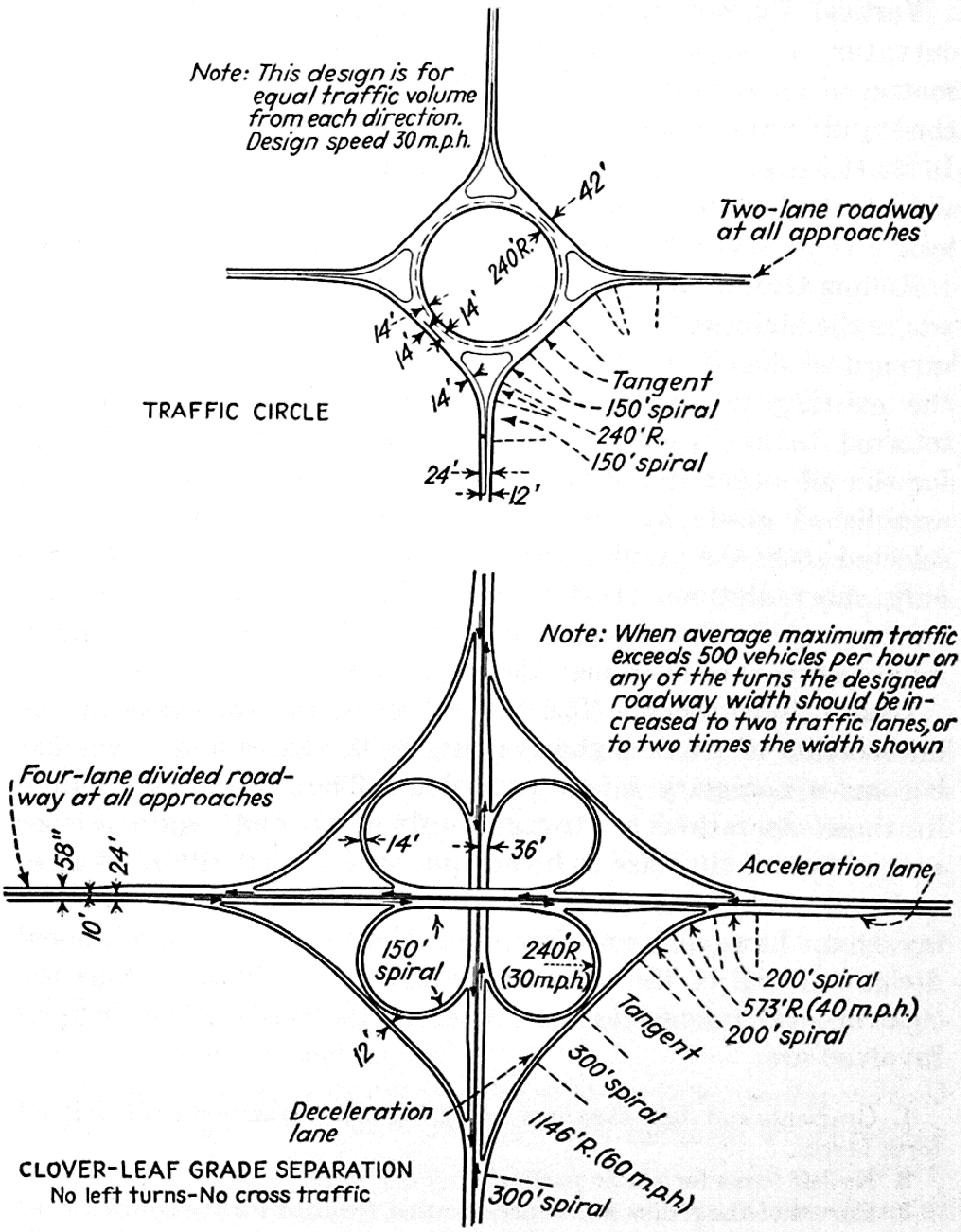

FIG. 40.—The traffic circle and the clover-leaf used at intersections.

The essential features of a clover-leaf intersection are shown in Fig. 40.

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BALANCING EARTHWORK

When grade lines are being established for proposed grade reductions, an attempt is made to balance the quantities of earthwork so that all excavated material will be used in the embankment. In order to accomplish the balancing, the quantities of earth in cut and fill must be calculated, and the probable amount of shrinkage, settlement, and subsidence estimated so that due allowance can be made for changes in volume resulting therefrom.

Computing Quantities.—In highway practice, the form of the final cross-section is such that the most convenient and accurate method of computing quantities of earthwork is by means of average end areas with cross-sections 100 ft. apart. Intermediate sections ("plus" cross-sections) are used for an occasional special condition, and in municipal work 50-ft. stations are frequently used for computing quantities. The crisscross method and other special ones are sometimes used for computing quantities, but most drafting rooms adhere to the end-area method.

Shrinkage and Swell.—When the volume of an embankment upon completion is less than the volume of the excavation from which the fill material was obtained, the difference is the *shrinkage*. If the volume of the embankment is greater than the excavation from which the material was obtained, the difference is the *swell*.

Most soils are sufficiently porous so that a cubic yard of excavated material will compact to somewhat less than a cubic yard of embankment because of shrinkage during the process of transporting and placing the soil in the embankment. The amount of shrinkage depends upon the nature of the soil, the size of the loads hauled from cut to fill, the prevailing weather, and the method of compacting. The actual shrinkage of ordinary soils, therefore, varies under differing conditions from 0 to 30 per cent or even more. The average, and quite general, allowance that is made for shrinkage of soils in balancing quantities is about 15 per cent, but the best practice is to estimate the shrinkage or swell by laboratory determination of the shrinkage limit and a study of actual shrinkage under compaction by the Proctor method (page 98) or some similar test.

Materials such as ledge rock, shale, and very hard-baked soils in desert regions will increase in volume when excavated and deposited in the usual manner. The extent of this increase in

volume will be from 10 to 50 per cent, depending upon the void content of the mass after loosening by explosives or otherwise. If an embankment is constructed of a mixture of soil and rock or shale, it will be necessary to determine whether the net result of mixing the two kinds of materials will be shrinkage or swell. This will depend upon the relative proportions of the two kinds of material and the nature of each.

The amount of earthwork shrinkage or swell depends not only on the sort of material that is being handled but also on the method employed for placing the material in the embankment. If the excavated material is handled in small loads by means of drag scrapers, the shrinkage will be the maximum, other factors remaining constant. Earth that is handled by power shovels and transported in dump trucks of several cubic yards' capacity will shrink relatively little. Between these two extremes lie several methods of excavation that produce intermediate degrees of shrinkage.

When fills of carefully graded materials are compacted by sprinkling and rolling to secure the maximum density, the shrinkage must be carefully estimated by laboratory studies of the materials proposed to be used in the work.

Settlement.—The decrease in height of an embankment that takes place subsequent to its completion is known as *settlement*. Embankments of earth generally settle slowly for many months after they are first completed, and the exact amount of settlement will vary with the nature of the fill material and the method of construction as well as with the weather conditions during and after construction. If the work is performed with the usual earth-handling equipment, the settlement will generally be about 10 per cent. If it is performed under conditions that cause a high percentage of shrinkage, the settlement will be slight, and fills that are compacted to maximum density as they are placed will not settle subsequent to completion. Rock fills, sand fills, and fills made up of mixtures of soil and a preponderance of rocky material will settle very little.

If a fill is expected to settle after completion, the finish grade stakes are set above grade to take care of the future settlement, and the fill quantities must be computed on the basis of the allowance made. For ordinary soils that allowance is about 10 per cent, but a greater or lesser allowance will be made in many instances.

Subsidence.—If an embankment is placed on a soil of low supporting strength, such as is found in swampy or near-swampy locations, or a very high embankment is placed on ordinary soil, the weight of the embankment may be sufficient to exceed the supporting strength of the soil of the site. In such instances the entire embankment subsides slowly, compressing the underlying soil or squeezing it aside. Whether or not this action will take place in any doubtful case cannot be determined readily, and the amount and rapidity of the subsidence can be judged only in the light of experience. Estimates of quantities of earth required for embankment on such sites are based on experience with similar sites and a guess at the exact condition of the site under consideration. Some means[1] may be taken to hasten the subsidence in swampy areas so that the fill can be completed under a single contract.

The urge in recent years to secure ample sight distance over crests has necessitated the construction of many fills of a height of 40 ft. or more. These frequently cause subsidence on what appears to be very good foundation soils. Evidence is accumulating that indicates such failures to be due to the weight of the fill causing shearing stresses in the foundation soil in excess of the strength of the soil in shear. If the strength of the soil in shear is determined before the grade line is established, the fill load can be held to a maximum that leaves a margin of safety against shear failure in the soil under the embankment.

Overhaul.—The estimates of the cost of excavation are generally based on specified length of haul, and to the basic cost is added a sum for each 100 ft. of haul distance in excess of the basic haul distance. The basic haul distance is generally called the "free haul" and is established by the terms of a contract in accordance with the common practice in the locality where the work is to be done. In most of the highway contracts the free haul distance is 500 ft.

The average excess length of haul is known as "overhaul." Thus if the average haul distance were 700 ft. and the free haul 500 ft., the overhaul would be 200 ft.

[1] Riedesel, P. W., "Blasting Settles Road Fills in Minnesota Muskeg," *Eng. News-Record*, Vol. 102, p. 788, May 16, 1929.

Cushing, J. W., and O. L. Stokstad, "Methods and Cost of Filling for Highway over Swamps," *Eng. News-Record*, Vol. 114, No. 4, p. 126, Jan. 24, 1935.

The computation of overhaul is not a simple matter, since the actual haul varies with the stage of the work and the method of excavation adopted. Overhaul is most accurately specified and computed from the mass diagram and should be so calculated when reasonably precise results are desired. Contracts should always define clearly the manner in which overhaul will be computed.

A good rule to use for approximate computations of overhaul is as follows: Lay off half the free haul distance in each direction along the center line from the point where the profile passes from cut to fill. Find the centers of mass of the remaining embankment and cut, and subtract from the distance between the centers of mass the free haul distance. This will be the overhaul distance. Subtract from the total excavation the quantity lying within half the free-haul distance from the point where the profile passes from cut to fill. This is the quantity overhauled. Multiply the quantity of overhaul by the overhaul distance in hundreds of feet and by the station-yard price of overhaul to obtain the overhaul cost.

Guard Fences.—Guard fences are placed along embankments of a dangerous height, along embankments on curves, and in other locations where it is desired to warn the traveler to be careful to stay on the traveled way. The value of the guard fence is partly psychological, because it is not customary to build the fence of sufficiently rigid construction to retain a heavy vehicle that might strike the fence when traveling at speed. Use of the guard fence has been greatly overdone, and there is really little excuse for it on tangent stretches of road—certainly not where the embankment is no more than a few feet high.

Guard fences are of many types for each of which there are many designs. One type consists of planking and posts; another of wire fencing and posts, usually with a plank rail at the top; and a third consists of posts and wire cable; still others utilize bands of sheet metal in combination with cables.

Guide and Warning Signs.—There is a growing sentiment in favor of excluding from the right-of-way of the highway all signs except the official ones provided by the highway authorities. Of these signs there are four distinct classes: (1) guide signs, (2) caution signs, (3) danger signs, (4) route markers. In some states each type has a distinct size and shape; in others all are

of the same shape and size except the route marker which is generally distinctive. Guide signs serve to indicate the distance and direction to various points and are placed near cities and at the junction of important roads. Caution signs are placed at the approach to cross-roads, curves, or other places where the driver should observe caution. Danger signs are placed at the approach to railroad crossings, turns, busy intersections, or other places of a similar nature. The difference between the caution sign and the danger sign is one of degree of danger only. Route markers are placed at frequent intervals along established routes and carry the route name and number and sometimes the mileage from the beginning of the route in the state or from some designated city.

These signs are best set on posts as near the traveled way as is safe, and the post should carry no information except the official sign.

Miscellaneous Appurtenances.—In many localities there is a goodly volume of pedestrian traffic on the public highways, and in the absence of any other provision for their convenience they will use the surface provided for vehicular traffic. This is objectionable because of the element of danger involved. A foot path should be provided at the outer edge of the shoulder to care for pedestrians and bicyclists, or it may be placed back of the ditch in level country. It may be of gravel, shale, or cinders and have a width of 3 ft.

Comparatively little attention is devoted by highway engineers to the provision of facilities for travelers who wish to camp along the roadside, since municipalities have generally arranged for tourist camps and have maintained them. The time will come when camp sites will be a part of the highway and will be so maintained. The requisites of a good camp site are (1) a well-drained, reasonably level area for camping purposes; (2) plenty of shade; (3) sanitary conveniences; (4) good water; (5) bathing facilities; (6) accessibility to all sorts of supplies and services. If the site is in or near a region of natural beauty, that enhances its desirability.

In addition to comfort stations provided in connection with camp sites there should be additional ones at rather frequent intervals along heavily traveled routes. All these facilities should be cared for by the patrolmen assigned to the adjacent section of highway.

STABILITY OF ROAD SURFACES

The fundamental principles underlying the combining of heterogeneous materials into road surfaces, foundations, and subgrades of the requisite durability have been sufficiently well established to permit general conclusions of considerable reliability. These principles are discussed at this place since they have a relation to much of the subject matter that follows and in which it will be convenient to employ frequently the term "stability" to designate a condition sought and the word "stabilized" to indicate the attainment of that condition.

Stability Defined.—The word stability is defined as "strength to stand or endure without alteration of position or material change." In this treatise the word stability is employed to indicate structural strength to endure the effects of climatic influences and the wear and weight of traffic loads without undue attrition, distortion, or rupture. Under an exact interpretation of the definition, nothing is stable, but some things possess a *high degree of stability,* whereas others have *little stability.*

Functions of the Road Surface.—A road surface may be thought of as a combination of a foundation course and a wearing surface, although in certain types of pavements the foundation and the wearing surface are monolithic and of identical composition. The traffic load is really carried by the soil subgrade. The road surface provides a medium that spreads the weight of the traffic loads over sufficient area of subgrade to insure a unit load that is within the safe load-carrying capacity of the soil. It also affords a wearing surface that is safe, durable, of acceptable riding quality, and renewable without disturbing the foundation course or, in the case of the monolithic types, capable of being resurfaced upon occasion. The foundation course and the wearing course each must possess sufficient stability to perform its intended function.

Degree of Stability Required for Wearing Surface.—The stability of the wearing surface should be sufficient to insure that the displacement of the wearing surface under wheel loads is well within the limits that experience has shown to be consistent with normal life for the surface. The criterion by which normal durability is judged is the annual cost of the surface, and the design attempts a balance between first cost and service life that will result in minimum *annual costs* per unit of service.

Degree of Stability Required for Foundation Courses.—The stability of the foundation should be such that the weight of the traffic loads will be carried to the supporting subgrade without harmful distortion of the foundation. The ability of the foundation course to transmit the load and at the same time distribute it over the required area of the subgrade is a function of the thickness of the foundation and also of its stiffness. Since the wearing course is interposed between the foundation and the wheel load, the foundation is not subjected to the direct contact of the wheel and the tendency to local distortion of the area of contact between wheel and surface. The foundation need not be constructed with a view to resisting wear (attrition), providing a dustless surface, or being of satisfactory riding quality.

Climatic Influences and Stability.—The probable stability of any road surface must always be appraised in the light of the possible effects of climatic influences upon the road surface and subgrade. Temperature cycles and surface and subsurface water from precipitation are the climatic factors involved.

Temperature cycles affect the stability of road surfaces in two ways. The temperature of the road surface structure will change slowly from hour to hour day and night, but the change will not be at one rate throughout the mass. Destructive strains may be set up within the road surface structure by temperature differentials. If the road surface is absorbent, and water is available to it, there may be internal strains of considerable significance due to the effect of the absorbed water. If, in addition, the weather cycle is such as to subject the absorbed water to cycles of freezing and thawing, additional destructive strains may be set up within the road surface structure.

The stability of the subgrade that carries the road surface is affected by an excess of absorbed water which serves as an internal lubricant and by freezing and thawing cycles when there is absorbed water, but the subgrade is not of a character to be influenced by internal strains set up by temperature differentials.

Traffic Influences on Stability.—A rolling wheel produces a kneading action which tends to cause distortion of the wearing surface in the area of contact between tire and surface and adjacent thereto. The material directly under the wheel tends to flow from under the load to which it is subjected. The wheels of self-propelled vehicles not only roll but slide as well. The driving wheels slide in propelling the vehicle; the steering wheels

slide in deflecting the vehicle on curves; all the wheels slide when the brakes are applied. The sliding subjects the surface to a scuffing or shearing action which imposes a horizontal thrust, often of considerable magnitude, on each particle of material exposed to contact with the tire, which tends to dislodge the particles and thus wear away the surface bit by bit.

The foundation course is subjected to distortion by the weight of traffic, which introduces into the structure tensile, compressive, and torsional strains that may be additive or compensatory to those induced by climatic factors.

The Nature of Mechanical Stability.—A box of small marbles of uniform size would possess little stability under the thrust of a pencil and still less if the marbles were immersed in water. If the marbles were covered by a stiff bit of board trimmed to fit loosely in the box, and a load were applied to the board, the mass of marbles would possess great stability. A paving brick, which is made from the same raw materials as the marbles, has great stability, wet or dry. The mass of marbles when unconfined has low stability because of low cohesion and low internal friction. The brick is highly stable because of high internal friction and high cohesion due to the particles of clay being melted in the burning. A layer of gravel pebbles of assorted size is of low stability under a wheel load, but a layer of broken stone of assorted sizes is of somewhat higher stability under a wheel load because of the high frictional resistance between the pieces of rock.

The Law of Granular Stability.—The maximum physical stability to be obtained in a mass of assorted sizes of any granular material is attained when the proportion of each size has been adjusted to secure the lowest void content possible with the range of sizes available. The maximum stability obtainable, measured by the ability of the mass to support loads applied on a relatively small area, depends upon the magnitude of the internal friction and cohesion. This law appears to apply to granular materials irrespective of the size of the largest particle, but the minimum void content that can be reached will depend upon the range of sizes available. It follows from the foregoing that the maximum mechanical stability that can be secured by combining two or more granular materials is reached when the amounts of each are so adjusted that the voids space is reduced to the lowest percentage obtainable with those materials. Such

a material is said to be "graded"; and if the operation has been done properly, is said to be "well graded."

This general principle of mechanical stability is utilized in stabilizing natural soil roads; in proportioning the sand and clay in sand clay and topsoil roads; in combining coarse sand, fine sand, and soil ("clay") in gravel road construction; in fixing the proportions of filler, fine sand, coarse sand, and coarse aggregate for bituminous concrete surfaces; and in proportioning the filler, fine sand, and coarse sand for sheet asphalt surfaces. It is also utilized in selecting the combination of the various sizes of sand and coarse aggregate to provide portland-cement concrete of the desired strength at minimum cost. The conception of portland-cement concrete as a mixture of mortar and coarse aggregate, although no longer considered valid in the higher technical circles, is so widely held that it is employed to establish an analogy that runs through the composition of all of the types of road surfaces. In stabilized soil surfaces two or more soils are combined to produce an acceptably low void content. A well-graded soil of the character thus secured is called "soil mortar."

Sand-clay surfaces consist of soil mortar and well-graded coarse sand.

Gravel roads consist of well-graded gravel and soil mortar or, more specifically, graded gravel pebbles, graded coarse sand, and soil mortar.

Portland-cement concrete consists of portland cement and graded sand (portland-cement mortar) and graded coarse aggregate (pebbles, crushed pebbles, crushed stone).

Bituminous concrete consists of bituminous cement, finely divided mineral matter, graded sand (bituminous mortar), and graded coarse aggregate.

Sheet asphalt consists of asphalt cement, finely divided mineral matter, and graded sand (bituminous mortar).

In all the foregoing it would be more precise to think of the surface as consisting of graded granular material and a binding agent, but the binding agent is not apparent in the description of the soil and gravel types.

Methods of Supplementing Mechanical Stability.—The degree of stability that it is possible to obtain by an economically obtainable mixture of granular material employed for a road surface may be insufficient to withstand the climatic influences

of the region and carry the traffic. Resort must then be had to the introduction of some binding or cementing agent into the mass of granular material to supplement the internal friction of the mixture. Two general methods are employed.

1. There is introduced in the mass a cementing agent of the nature of a slightly plastic glue. The bituminous materials are the only materials of this character so far utilized for this purpose. These materials are capable of being dissipated in extremely thin films which adhere to the surfaces of the aggregates and very greatly increase the cohesion and internal friction in the mass above that of dry granular materials. The road surfaces constructed of these materials are called the "flexible types" because they withstand a certain amount of distortion without rupture.

In mixtures of soils the very fine particles form with the moisture drawn into the soil a sort of colloidal glue whose cementing action depends primarily on the surface tension of the extremely thin film of water surrounding each particle. A deliquescent salt may be incorporated in the mixture to assure there being enough moisture attracted to the mixture.

2. There may be introduced into the mass of granular material a solution or paste from which are deposited crystals of mineral matter on the particles of aggregate, thus bridging the space between particles and binding the grains together with crystals that, in the case of portland cement, have great strength. A similar result is obtained in water-bound macadam construction by the puddling process employed in finishing the surface. Here the crystals formed are of nowhere near the strength of those formed in portland-cement mortar.

The Mechanics of Loss of Stability.—When a mass of granular material, whose natural mechanical stability has been increased by means of some binding or cementing agent, is subjected to disruptive forces or climatic influences of such a character as to cause harmful distortion or even rupture, the failure may be explained by the behavior of the cementing agent under the strain to which it was subjected.

1. The destructive force to which the flexible types of surfaces are subjected may be so great that the cohesion and internal friction are insufficient to restrain the movement of the granular particles. The soil-mortar-bound surfaces depend for their resistance to displacement upon the surface tension of a moisture film that has low resistance. The bituminous-bound surfaces

TABLE XIV.—CLASSIFICATION OF ROADS AND REQUIREMENTS FOR DESIGN

	Class *A*	Class *B*	Class *C*	(Minor trunk line) Class *DE*	(Local roads) Class *E*
Traffic	4,000 or more	750–4000	300–750	300 max.	200 max.
Surfaced width	More than 2–10 ft. lanes	2–10 ft. lanes	20 ft. min.	20 ft. min.	9 ft. min.
Shoulders or roadbed width	8 ft. min. clear surfaced or unsurfaced shoulders	Same as Class *A*	6 ft. clear surfaced or unsurfaced shoulders	24–28 ft. roadbed	20–24 ft. roadbed
Alignment (between control points)	Straight as feasible 2° 30′ approx. max. 5° max.	Same as Class *A*	Straight as feasible 4° approx. max. 8° max.	Straight as feasible	Local standards
Sight distance (with 5 ft. height)	800 ft. min. more if feasible	Same as Class *A*	800 ft. min. to be used where alignment permits high speeds. 500 ft. min. except in difficult topography 350 ft. min.	350 ft. min.	Local standards
Grades	5% max.	5% max. where feasible. Where topography requires up to 8% permissible	6% max. where feasible. Where topography requires up to 8% permissible	9% max.	Best feasible
Surface types	Pavement	Pavement	Dustless all-weather surface.	All-weather surface	All-weather surface
Right-of-way	100 ft. min. for ultimate 4-lane traffic; 120 ft. min. for ultimate 6-lane traffic	80–100 ft. min.	80–100 ft. min.	80 ft. min.	Local standards
Structures over 20-ft. clear span	Clear roadway 4 ft. wider than ultimate pavement	Same as Class *A*	24 ft. min. roadway	24 ft. min. roadway	20 ft. min. roadway
Structures less 20-ft. clear span	Full width of roadway	Same as Class *A*	Same as Class *A*	Same as Class *A*	Same as Class *A*
Railroad grade crossings	Eliminated if practicable, otherwise protected with automatic or manual device	Same as Class *A*	Protected or eliminated	Protected or eliminated	Permitted

depend upon the much greater surface tension of the film of bituminous material, which, however, is of relatively low resistance to displacement.

TABLE XV.—TRAFFIC DENSITY COUNTS

Town	Location	Vehicles per hour per lane
1. Washington, D.C.	Constitution Ave. at 12th	310
2. Marshalltown, Iowa	Main St.	136
3. Des Moines, Iowa	Fifth and Court	508
4. Pittsburgh, Pa	Downtown Street	754
5. Fort Dodge, Iowa	12th St. at 2d Ave. S.	172
6. Cedar Rapids, Iowa	1st Ave. at 13th St. E.	175
7. Detroit, Mich	Grand Blvd. at 14th	550
8. Newark-Jersey City, N.J	N.J. High-level Viaduct	1,100
9. Cleveland, Ohio	Rocky River Bridge	625
10. St. Louis Co., Mo	Big Ben Road, east to city limits	233
11. St. Louis Co., Mo	Ferguson, S.N.	375
12. Baltimore-Washington Highway	....	1,502
13. Chicago, Ill	Lake Shore Drive	1,349
14. Cleveland, Ohio	Superior-Detroit	1,241
15. Detroit, Mich	High-level Bridge	1,557
16. Detroit, Mich	East Grand Blvd.	1,200
17. New York, N.Y	Holland Tunnel	1,253
18. New York, N.Y	Queensborough Bridge	1,482
19. New York, N.Y	Manhattan Bridge	1,300
20. U.S. 20	2.4 miles west of Norwalk, Ohio	850
21. U.S. 23	1.0 mile west of Delaware, Ohio	920

The bituminous-bound mixtures may also fail because the film of binder no longer adheres to the aggregate, which may be due to a film of water having formed between the bituminous binder and the aggregate. In some instances the character of the surface of the aggregate may be such that the binder does not readily adhere to it.

2. Surfaces consisting of granular material held together by interlaced crystals like portland cement concrete and water-bound macadam may fail because the crystals rupture or because the crystals break loose from the aggregate to which they are

attached, or the aggregates themselves may break. The water-bound macadams fail by rupture of the crystals that serve as binder; the portland-cement concrete probably fails from a combination of all three actions, although that has not yet been proved.

TABLE XVI.—RECOMMENDED LENGTHS FOR SPIRAL TRANSITION CURVES

Main trunk highways				Secondary highways				Mountain roads			
Degree of curve	Radius, feet	Speed, m.p.h.	Length of spiral, feet	Degree of curve	Radius, feet	Speed, m.p.h.	Length of spiral, feet	Degree of curve	Radius, feet	Speed, m.p.h.	Length of spiral, feet
1	5,730	80	141	2	2,865	60	119	3	1,910	45	75
2	2,865	80	282	4	1,433	60	238	6	955	45	150
3	1,910	80	424	6	955	60	357	9	655	45	220
4	1,433	80	565	8	716	60	476	12	478	45	302
5	1,146	80	706	10	573	60	595	15	382	45	377

Applications to Highway Construction.—The numerous types of roadway surfaces encountered in the United States as well as elsewhere represent the results of efforts to produce durable wearing surfaces out of the materials most readily available and at costs consistent with the budget of the responsible authority. A careful study of the specifications for each type will be convincing evidence that empirical methods of combining materials are used more frequently than scientific methods that take account of the fundamental principles of proportioning for the required degree of stability.

It is recognized by competent highway departments that the required degree of stability can be ascertained only by a study of the climatic influences in the area and of the volume and weight of the traffic that will be likely to use the road surface after completion.

In later chapters the general principles that have just been outlined will be applied to each of the classes of roadway surface in common use, along with a discussion of the influence of the local climatic and traffic conditions upon construction practice in the region.

TABLE XVII.—AVERAGE COEFFICIENTS OF FRICTION OF WET ROAD SURFACES

Type of surface	Skidding straight ahead						Skidding sidewise					
	Speed and condition of tires						Speed and condition of tires					
	New	Worn smooth	New	Worn smooth	New	Worn smooth	New	Worn smooth	New	Worn smooth	New	Worn smooth
	10 m.p.h.	10 m.p.h.	30 m.p.h.	30 m.p.h.	50 m.p.h.	50 m.p.h.	10 m.p.h.	10 m.p.h.	30 m.p.h.	30 m.p.h.	50 m.p.h.	50 m.p.h.
Asphaltic concrete	0.80	0.80	0.60	0.65	0.50	0.55	1.00	0.90	0.92	0.84	0.80	0.75
Asphalt, sheet	0.80	0.86	0.65	0.50	0.45	0.40	0.93	0.83	0.91	0.72	0.84	0.63
Asphalt, rock, sandstone, sand	0.90	0.86	0.75	0.55	0.45	0.30	0.93	0.89	0.88	0.70	0.76	0.52
Asphaltic macadam retread	0.75	0.55	0.55	0.40	0.40	0.32	0.71	0.73	0.62	0.61	0.58	0.58
Brick, grout filled	0.60	0.50	0.50	0.38	0.47	0.30	0.61	0.52	0.58	0.40	0.56	0.30
Brick, asphalt filled	0.70	0.60	0.52	0.41	0.50	0.32	0.57	0.60	0.52	0.47	0.48	0.42
Concrete-portland cement	0.65	0.60	0.50	0.40	0.40	0.35	0.75	0.68	0.62	0.50	0.52	0.40
Gravel, ordinary untreated	0.70	0.65	0.70	0.65	0.70	0.60	0.60	0.58	0.68	0.59		
Macadam, penetration, soft seal coat	0.52	0.45	0.30	0.25	0.30	0.18	0.76	0.63	0.62	0.37	0.58	0.30
Mud on pavement surface	0.25	0.20	0.25	0.20	....	....	0.23	0.25	0.23	0.20		
Medium tar surface treatment	0.65	0.70	0.47	0.42	0.40	0.25	0.90	0.80	0.72	0.67	0.58	0.50
Oiled gravel	0.70	0.70	0.48	0.43	0.43	0.35	0.85	0.72	0.68	0.53	0.53	0.40
Road oil mixture	0.72	0.65	0.50	0.36	0.45	0.30	0.87	0.80	0.78	0.60	0.64	0.54
Packed snow	0.10 or less											
Sleet	0.05 or less											

CHAPTER VII

THE DESIGN OF STREETS

There are some problems of design that are solved in the same manner, whether the improvement is for a rural highway or a city street, whereas others involve special considerations because of the differences in use. The design of street improvements, like that of rural highways, must be predicated on three kinds of traffic: vehicular, hydraulic, and pedestrian—with pedestrian traffic much more of a factor in the cities than it is in the country.

Functions of the Pavement.—The pavement on a city street serves two functions: It carries vehicular traffic, and it conducts drainage water from the street area and from the adjacent property to an inlet to the underdrainage system. The pedestrian traffic is, of course, carried by the sidewalks and cross-walks at intersections.

A few cross-sections, which indicate typical designs for various classes of streets, are shown in Fig. 41.

The typical cross-sections illustrate the normal pavement in which the cross-section is symmetrical except where warping is necessary to secure a neat connection at the intersections with the other streets. The pavement proper is flanked by curbs of equal height on the two sides of the pavement; the longitudinal grades of the center line of the pavement, the gutter line, and the top of the curb are all of the same slope.

On many projects it is impossible to employ the normal cross-section throughout, and certain acceptable deviations from the normal cross-section have come into general use. It may be necessary to shift the crown line of the pavement away from the center line, change the curb height from place to place, and carry the gutter grades on a slope different from that of the crown line. The pavement surface is said to be warped when the gutter grade line and the grade line of the middle of the pavement do not lie in a plane. Warped surfaces are quite common in paving practice.

Certain fairly definite principles underlie and limit these departures from the normal if the design is well conceived, and

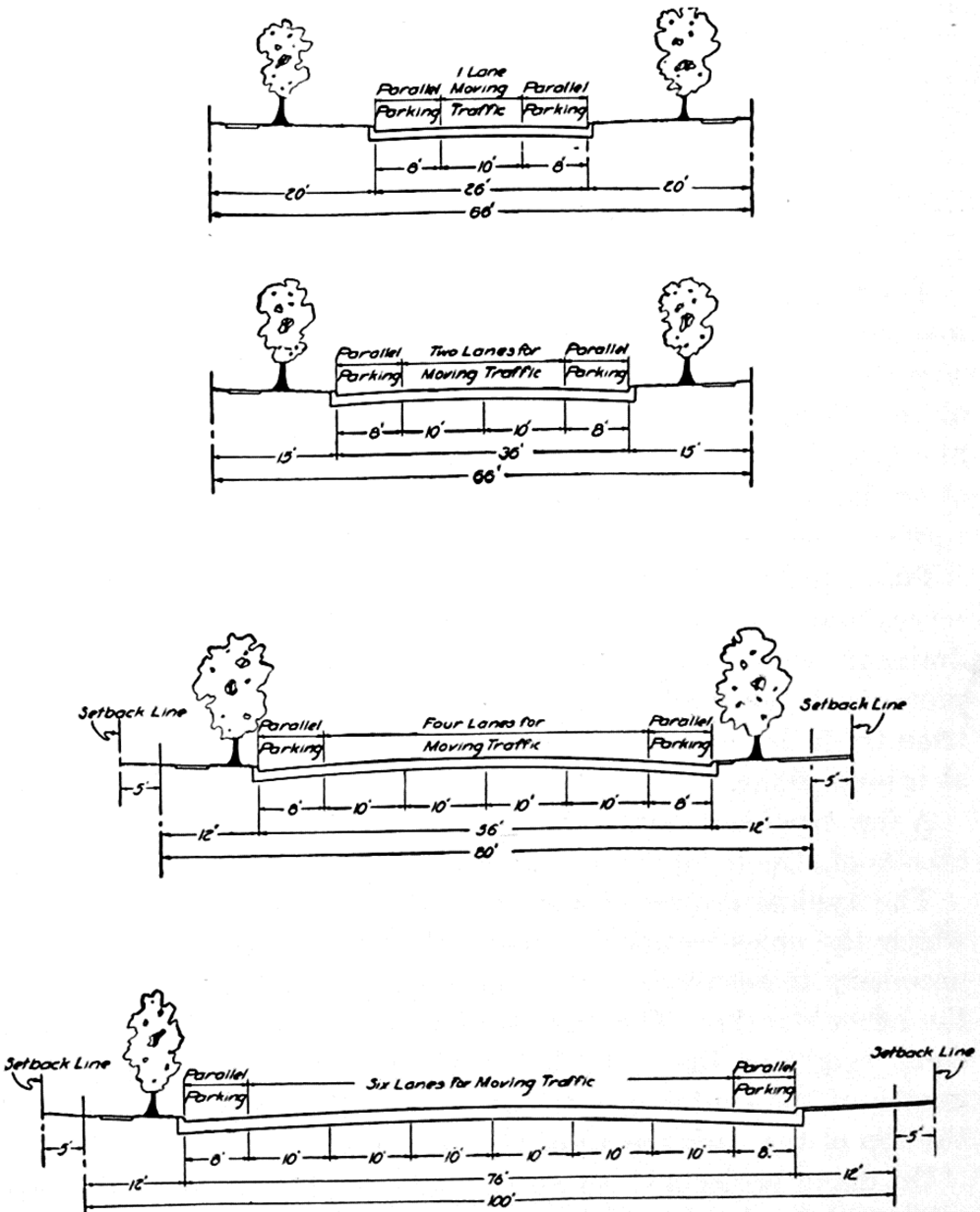


FIG. 41.—Types of cross-section suggested by Chicago Regional Planning Commission.

these will be pointed out in the discussion that follows.

Elements of Design.—The design of street improvements involves separate consideration of a group of related problems. The more important are:

1. Determination of width of pavement.
2. Establishment of street grades.

3. Design of crown for pavements.
4. Layout and design of intersections.
5. Design and location of drainage appurtenances.

WIDTH OF PAVEMENT

The determination of the width of pavement for any location will be based upon the traffic capacity needed, the width of right-of-way, and the financial resources, important in the order in which they are listed.

Traffic Capacity Influence.—Although traffic capacity is the first consideration, it is the most illusive factor with which the designer will deal. A method of estimating the traffic capacity of a rural highway was discussed in Chap. VI, but it was pointed out that the method yielded only approximate results because of the difficulty of fixing average speed and headway. The same difficulties exist when dealing with estimates of the traffic capacity of the city pavements. In addition, the influence of cross-traffic, vehicles entering or leaving parking places, bus and truck traffic, pedestrian traffic at intersections, and traffic regulation systems all enter to complicate the problem.[1] Traffic counts have been entirely too few to permit generalizations, and because of the high cost of the municipal traffic census there seems little probability that data on actual traffic capacities will accumulate very rapidly. Studies of traffic capacity that have been reported indicate that the maximum discharge per traffic lane, under good conditions of regulation and on arterial streets with good pavements, will exceed 1,000 vehicles per hour, whereas the theoretical capacity is at least 2,500 vehicles per hour. It seems to be the rule that it is only in the event that important arterial streets are being developed that there is any systematic attempt to estimate the traffic capacity of proposed improvements.[2]

In certain cases it is possible to estimate from the general character of a street the number of traffic lanes that will be required, which is the best that can be done unless definite data on traffic are available. It will be convenient to consider the following typical kinds of streets.

[1] LEWIS, HAROLD M., "Metropolitan Traffic Control," *Roads and Streets*, Vol. 65, No. 5, p. 277, May, 1926.

[2] KELKER, R. F., JR., "Capacity of Roadways," *Roads and Streets*, Vol. 65, No. 1, p. 9, July, 1926.

1. Residential streets carrying little through traffic.
2. Residential streets carrying through traffic.
3. Streets in retail business areas.
4. Streets in wholesale and manufacturing districts.
5. Boulevards or arterial streets with traffic restrictions.
6. Arterial streets for mixed traffic.
7. Effect of car lines on required width.

Local Streets.—The simplest case is that of a local street intersected by through streets that permit traffic to move away from the district. These local streets may be built up with individual residences or with apartments and scattered small shops. The minimum width permissible is that which will provide three traffic lanes, two of which may be occupied at times by standing vehicles, leaving one clear lane for moving traffic. On the usual basis of 10 ft. as the proper width for a traffic lane, such a street would have a pavement 30 ft. wide. In outlying areas of low valuations and very light traffic, this might be reduced to 26 ft. If apartment houses and high-class individual residences predominate, the ordinary minimum width is four lanes, or 40 ft. This might be reduced to 36 ft. in exceptional, or intermediate, cases. The presence of standing traffic must be accepted as inevitable on such streets, and room for moving traffic provided in addition to that required for standing or "parked" vehicles.

Through Streets in Residential Districts.—Streets of this type carry not only the traffic that serves the residences on the street and in the vicinity but also appreciable amounts of traffic having origin and destination outside the district—through traffic. The minimum width for such streets is 40 ft., or four lanes, which is the prevailing width of streets of this class. In some instances, the through traffic requires more than two lanes, and yet the street width will not permit the construction of a 60-ft., or six-lane, pavement, and a compromise is reached by making the width 50 ft. This provides scant room for four lanes of moving traffic if the standing vehicles are placed parallel to the curb and close thereto.

Retail Business Streets.—Streets in retail business districts range all the way from the typical "main street" of the small city to those in the great department-store districts of the larger cities.

The problem of design in any district of this sort is to divide the available space between the vehicular traffic and the pedestrian traffic. The entire width of the street is utilized for side-

walks and pavement, and there is no fixed rule for the division of the space that is universally applicable. It seems to require from one-third to one-half of the total street width for sidewalks, and, in general, the higher proportion of sidewalk space is required in the districts devoted to the larger retail establishments. The minimum width of pavement is 40 ft. regardless of the size of the city, and in cities of a population of 15,000 or more the pavements should be wider than 40 ft. The retail- and department-store districts of the larger cities present a difficult problem in this respect, because curb lines have been long established, and building lines can be moved back only at tremendous cost. Therefore, little change in pavement widths is to be expected in such districts, but minor changes are worked into the layout from time to time by way of correcting glaring inadequacies, either in pavement width or in sidewalk space.

Warehouse and Wholesale District Streets.—Districts devoted to wholesale business and warehouses have need for all the pavement area that can be provided and relatively little need for sidewalk space. On account of the space required by vans and trucks that are drawn to the curb to receive or discharge cargo, the standing traffic will require about 20 ft. along each curb. Two lanes for moving traffic will add 20 ft., making the total width of pavement 60 ft. This is the usual minimum. There are many variations of the problem of width in districts of this character. Sometimes the sidewalk is omitted entirely so as to secure enough pavement area for vehicular traffic. In other areas, the presence of factories necessitates a reasonable amount of sidewalk space. There are districts where the traffic cannot be accommodated if trucks are permitted to load and unload at the curb, so the business houses are required to provide space within the buildings for the standing traffic. Sometimes the alley is used for the purpose, but generally alleys do not provide enough room and must be supplemented by space inside the building line.

Street-car Tracks.—The actual traffic width required for a single-track car line is about 10 ft.; and for a double-track line, 16 ft. The space occupied by the car tracks is used by vehicular traffic, and no estimate of the extent to which the traffic capacity of the street is reduced because of the street-car traffic is of any particular value. As a minimum, the pavement should be wide enough to provide for one lane of moving traffic and one

of standing traffic on each side of the area used for street-car movement.

LONGITUDINAL GRADES

The term "grade" is used herein as meaning the slope in the longitudinal direction of the pavement, as along the gutter or center line. Grade is usually expressed in per cent, which is the change in elevation in feet in 100 ft. of horizontal distance. The term "elevation" is used to indicate the heights above datum of the various parts of the pavement. Elevation and grade are often used synonymously in engineering practice; hence the preceding definitions.

A grade line is a line (usually on a drawing) connecting the elevations of points on the pavement, curbs, or walk that are selected as control points for design or construction. There may be a grade line for the center line of the pavement, for the gutter, or for the top of the curb.

The principles outlined in Chap. V apply to grades on streets as well as on rural highways, except that traffic conditions generally preclude taking advantage of momentum grades on city streets. The theory of grades is of service in fixing maximum grades, but even in this respect topographical limitations often govern, rather than traffic convenience.

Maximum and Ruling Grades.—Since the designer is interested only in grades for improved streets, the maximum grade for an automobile street should be that which will permit automobiles to ascend at constant speed without shifting gears, and with the mixture of models of automobiles in use in 1938 this is about a 7 per cent grade. If commercial vehicles must be taken into account the grade should not exceed 4.5 per cent. Keeping in mind the fact that descending vehicles must use the brakes on grades of more than about 4 per cent, it follows that the ruling grade should be as near 4 per cent as can be secured, with 7 per cent the limit. Topographical considerations will generally determine the actual maximum grade to be used.

The type of surface to be used must be considered in determining the limiting grades; or if topographical conditions preclude grade adjustments, the streets must be paved with a material suitable to the grade. If it is necessary or desirable to use a certain type of pavement, such as bituminous concrete, then the

grades must not exceed that which experience has shown to be the maximum permissible for that type.

Table XVIII gives safe limits of grade for the various types of surface if they carry mixed traffic; but as may be expected, there is little uniformity in practice in this respect.

TABLE XVIII.—ORDINARY MAXIMUM GRADES FOR VARIOUS TYPES OF SURFACE

	Per Cent
Asphalt block	6
Brick	10
Sheet asphalt	5
Bituminous macadam	8
Bituminous macadam without seal coat	10
Concrete	8
Hillside brick	12
Bitulithic, asphaltic concrete	7

Minimum Grades.—The minimum grade is determined by hydraulic rather than vehicular traffic. The gutter along the pavement carries drainage water to the inlets to the sewers, and the minimum grade is that which will be sufficient to carry the water without accumulations. This is about 0.3 per cent grade for concrete or other smooth gutters and 0.4 per cent for cobble or other rough gutters.

The grade of the crown line of the pavement is sometimes different from that of the gutter or the top of the curb. This scheme is often employed for streets in areas that are naturally about level and requires that the cross-section of the pavement change gradually from one of minimum cross-slope at the crests in the gutter grade to one of maximum cross-slope at the storm-water inlets. The exigencies of fitting pavement grades to existing sidewalks or drainage systems also require that in some cases the gutter grade be at a different rate from the crown-line grade.

Hence it follows that the curb grade and the crown-line grade may each be 0.0 per cent and the gutter grade the minimum of 0.3 per cent.

Curb Elevations.—The gutters along the pavement are the primary drainage channels for the water flowing off the adjacent properties, and the first principle of fixing curb elevations is to insure that such water can flow over the curbs. The ideal arrangement would be to provide a slope upward from curb to

sidewalk of not less than 2 per cent or more than 3 per cent and a like slope from the sidewalk to the building line. Good practice adheres rather closely to this ideal for the space between the curb and sidewalk, but the slope of the lawn is a matter for the individual owner, and no common practice can be said to exist. Sidewalks are generally constructed before a street is paved, and the lawns of residential property and the thresholds to the entrances to business property are adjusted to the sidewalks. When sidewalks have been constructed to a carefully established system of grades, it is comparatively easy to fix curb elevations, since the top of the curb must be at an elevation determined by the slope rule already mentioned. It frequently happens that sidewalks have been constructed without regard to a system of grades, and curb elevations will then be established so as to permit storm water to flow to the pavement throughout the length of the street, although some sections of low walk may have to be disregarded. Likewise, some building lots may be so low that it is impossible to fix curb grades so as to drain these lots, and the owner must fill them up before they can be drained. If sidewalks have been constructed on a street, the curb elevations should never be fixed until the adjacent walk grades have been considered.

The second principle to be observed in establishing curb elevations is to preserve the minimum grade requirements for the gutter, which in the normal case means that the curb grade should not be less than 0.3 per cent. If that cannot be accomplished because of the natural topography, then the gutter grades will of necessity be run independently of the curb grades.

The third principle to be considered in establishing curb grades has to do with the appearance of the completed improvement. It is that abrupt changes in curb grades should not be employed; and if straight-line grades are not practicable, then regular vertical curves should be employed. When straight-line grades are used, changes in rate of grade are less noticeable if they are made at street or alley intersections, where the continuity of the curb is broken. If the curb grade is made in the form of a vertical curve, the parabolic form is usually employed, and its length is based on securing safe sight distance and fitting the existing topography as closely as possible. Where the parabolic form is used, the necessary length is first determined, and the elevations of the P. V. C. and P. V. T. ascertained. The elevation of the

curve at the middle ordinate is then calculated from the properties of the parabola as follows:

$$E_3 = E - \tfrac{1}{2}[E - \tfrac{1}{2}(E_1 + E_2)]$$

where

E_3 = elevation at middle ordinate.

E = elevation at P. I. of the straight-line grades to be connected by the parabola.

E_1 = P. V. T., elevation at point of vertical tangency.

E_2 = P. V. C., elevation at point of vertical curvature.

Elevations of intermediate points are readily calculated from the mathematical properties of the parabola.

Relation of Pavement to Curbs.—The general grade of the pavement is fixed by the curb elevations, since the pavement elevations bear a definite relation to the curb elevations. Some leeway exists whereby the pavement elevations may be established independently of the curb elevations, but no major departure is permissible. It is best to think of the pavement and curbs as an entity and assume that whatever circumstance affects the grade for one also affects the grade for the other. In special cases, the height of curb may be varied somewhat, or the curb may disappear altogether for short distances, particularly near intersections, but it is better to carry pavement and curb elevations so as to produce the normal cross-section so far as possible.

Unsymmetrical Cross-sections.—When the property on one side of a street is higher than on the other, the curb on the higher

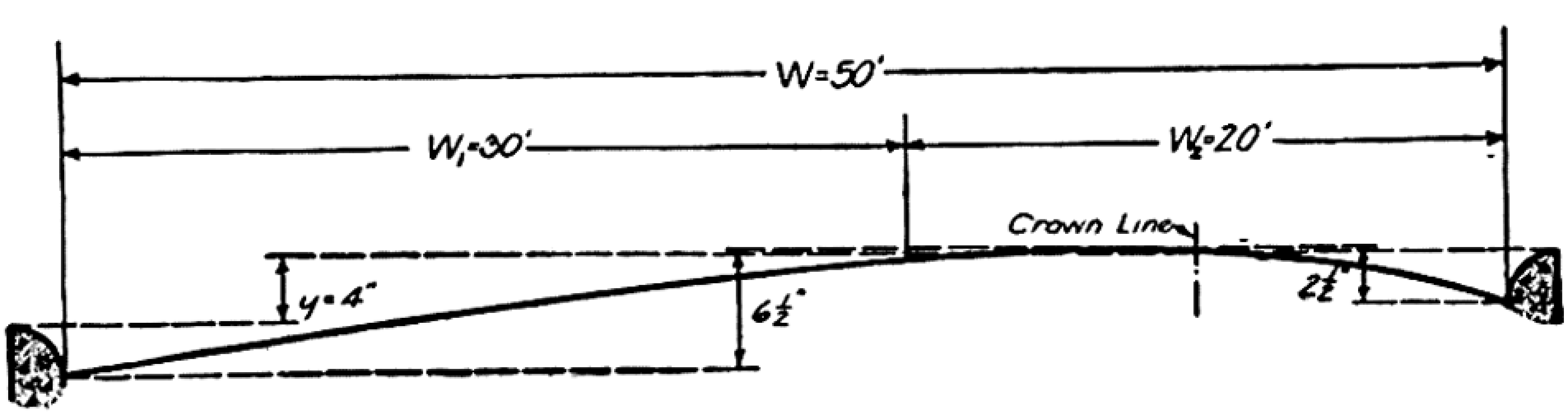


FIG. 42.—Illustrating design of unsymmetrical cross-sections.

side of the street will be set at a higher elevation than the curb on the low side, to give a good appearance to the street and to minimize grading. This plan also serves to simplify the design of intersections on the street. The maximum permissible difference in the elevations of curbs in such cases is equal to the total crown for a pavement of double the width of the pavement in question.

If the difference in curb elevations is less than the maximum that is allowable, the cross-section is designed as follows (see Fig. 42):

1. Determine the width of pavement for which the difference in curb elevation would be the correct total crown, and divide by two, and designate this as W_1.

2. Let W designate the total width of the pavement, then $W - W_1 = W_2$, the width of pavement assumed to have the normal crown. The correct crown for a pavement of width of W_2 is determined from one of the crown diagrams.

3. The crown line will be at a distance $\frac{W_2}{2}$ from the higher curb or a distance of $W_1 + \frac{W_2}{2}$ from the lower curb, and the pavement surface on each side of the crown line will conform to a parabola with its vertex at the crown line and passing through the gutter elevation.

CROWN

When a pavement is constructed with a normal cross-section, the surface is convex in form so that storm water will flow to the gutters.

No very rigid rules can be laid down to govern the amount of crown to use, and there is not close agreement among engineers with respect to the amount of crown and the form of the transverse section.

Purpose of Crown.—Pavements are constructed with crown for drainage purposes, and therefore the amount and form of crown are determined by drainage requirements. If the rate of longitudinal slope is small, the cross-slope must provide the drainage. The amount of cross-slope is therefore made sufficient to cause the storm water to pass quickly to the gutter. The smooth and impervious types require less crown than those types which may be somewhat absorbent or tend to become slightly uneven as they are used. The differences are so slight among the high-type pavements that it seems scarcely worth while to vary the crown with the type. If the longitudinal grades exceed 2 per cent, the cross-slope may be reduced somewhat from the rate used on pavements on flat grades.

In some climates the drainage requirements do not necessitate more than a semblance of crown, but the pavements look better if they are crowned a little, and also they will be easier to clean by flushing than if built without crown.

Form of Crown.—It has long been the custom to use the parabolic form of crown for pavements, and in general it is satis-

factory, except that for wide pavements, for example those 60 ft. or more in width, the parabolic form has been criticized because the central portion will be nearly flat. On a 50-ft. street with a total crown of 6 in., the total crown of the central 25 ft. would be 1.5 in. if the parabolic form is used. Some designers feel that to be too little and prefer to use a form of curve that gives a little more cross-slope to the central part of the pavement.[1]

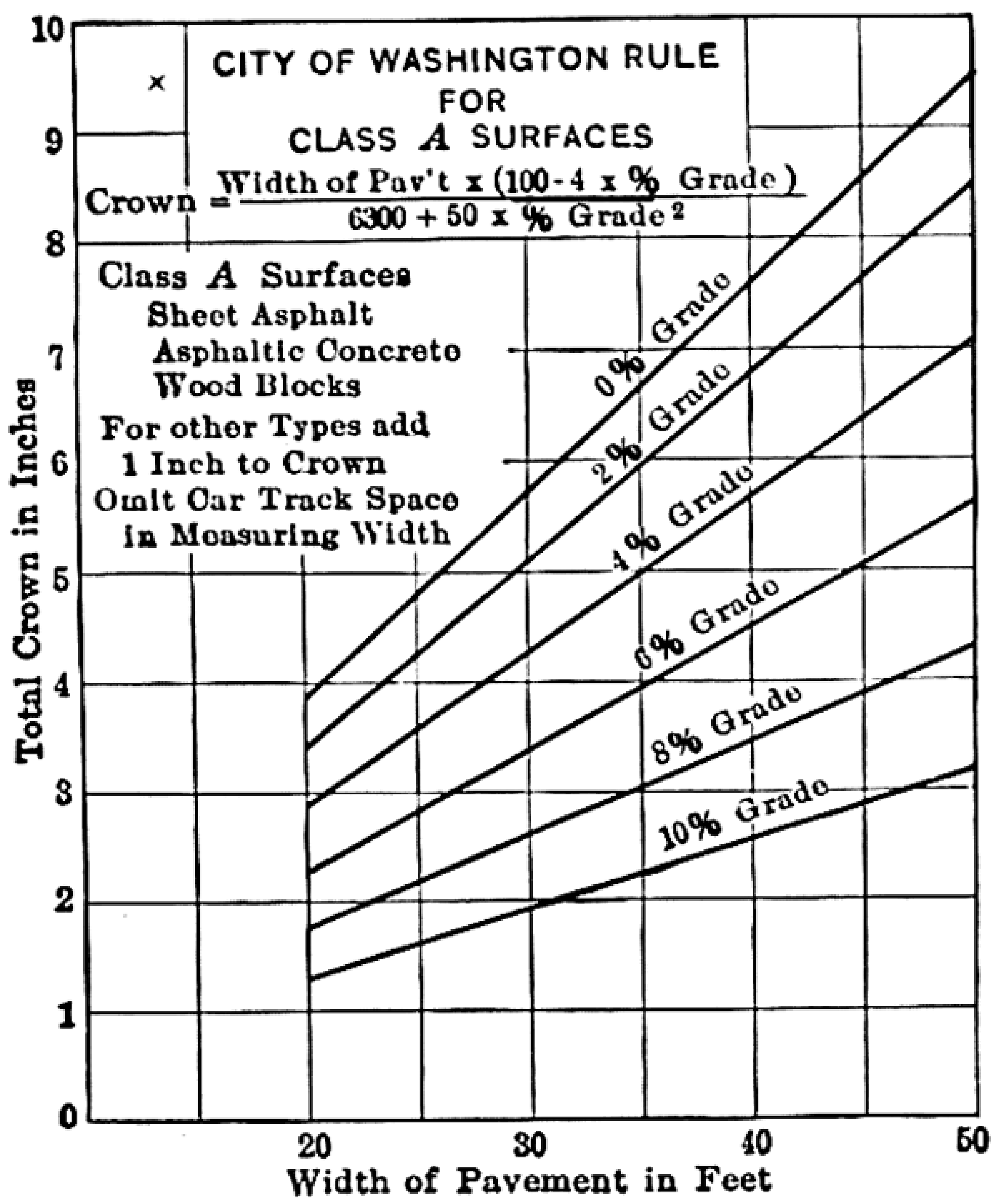


FIG. 43.

Amount of Crown.—There are in use a number of rules for determining the total crown to use for streets of various widths and grades. These are empirical in character but have had wide use and have proved acceptable. This is especially true of the Rosewater rule, developed first for use in Omaha, Neb. The author has used a simple rule which seems to have been quite satisfactory but provides for somewhat less crown than some of the others and is believed to be advantageous for motor traffic where the problem of skidding must be considered. In Figs. 43–45 are several diagrams for determining crown in accordance

[1] BESSON, F. S., "Methods for Determining Street Pavement Crowns," *Eng. News-Record*, Vol. 90, No. 14, p. 627, Apr. 5, 1923.
ROSS, ERNEST S., "Designing Pavement Crown," *Eng. News-Record*, Vol. 99, No. 4, p. 152, July 28, 1927.

with the practice in certain cities which have standardized on certain rules that have been applied with acceptable results.

INTERSECTIONS

The design of the intersection of two streets has considerable influence upon the grade line of each of the streets between inter-

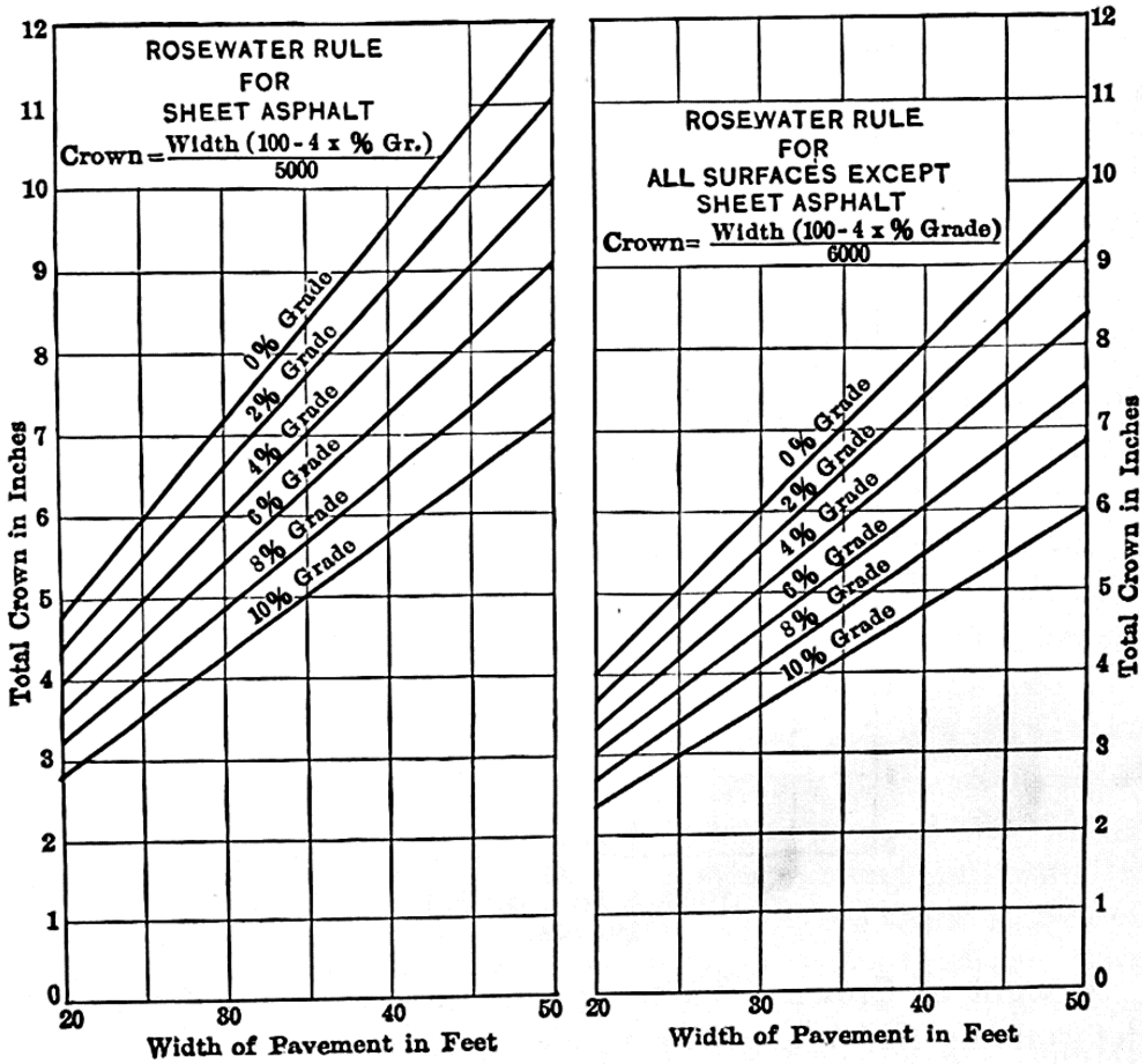


FIG. 44.

sections, particularly when the rate of grade on the streets is such as to require flattening of grades across intersections. The design of intersections involves determining the type of treatment in general and the establishment of the exact elevations of the curbs and pavement at control points in the intersection and adjacent thereto.

Types of Intersection.—Intersections are of two general types, but the two are not so very clearly defined in practice, and it is not uncommon to see intersections where one or two corners are of one type and the remainder of the corners are of the other.

It is convenient to discuss the two as distinct types, which for convenience are designated *A* and *B*, even though many variations from type may be expected.

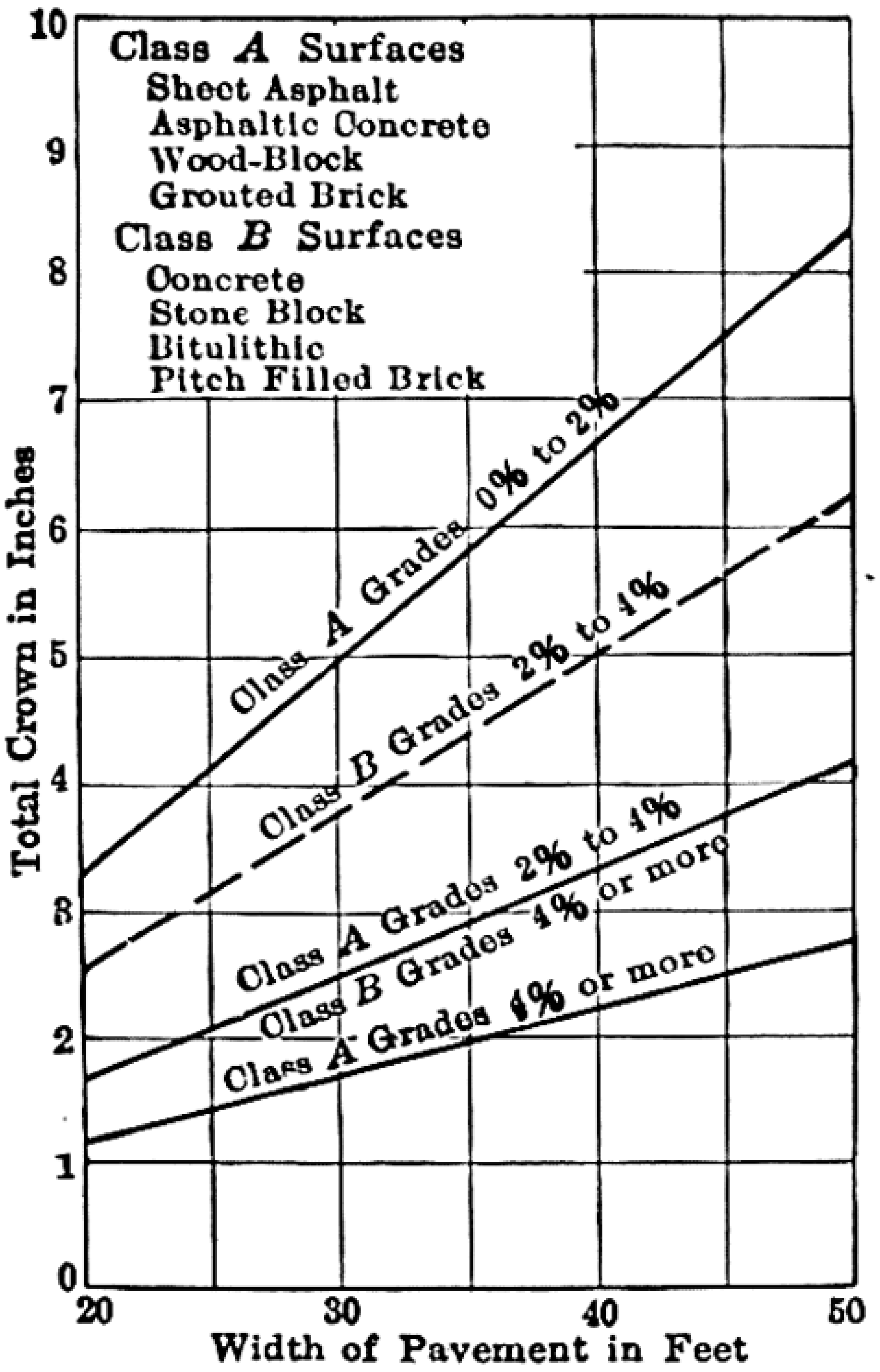


FIG. 45.

Type A.—This type is designed with normal curb exposure through the intersection. At each pedestrian crossing there will be a step equal to the curb height and a small flow of water near the curb during periods of precipitation. This design is used in districts where the space between the curb and the building line is entirely occupied by sidewalks and in other districts where vehicular traffic is heavy and pedestrian traffic is also fairly heavy. It is intended to keep the vehicular traffic strictly to its own area, thus protecting pedestrian traffic. An intersection of this kind is shown in Fig. 46.

Type B.—Another type of intersection is designed with the pedestrian crossing raised to the elevation of the top of the curb. It is intended to eliminate the step at the curb and, in a measure, to afford a dry crossing for pedestrians in wet weather. It is a type that is extensively used in residential districts and in the less congested business sections of the cities. In some cities it is used throughout the city on the projects of recent design. There are many differences in the details of the designs of intersections of this type, but all seek to accomplish the same thing. One development of an intersection of this kind is shown in Fig. 47.

Curb Radii at Intersection.—The radius of curvature of the curbs at intersections is made as great as is possible without

encroaching on sidewalk space. In residential districts where there is a width of turf or an unused space between the curb and the outer edge of the walk, the curb radius is fixed at about 20 ft. If a curve of 20 ft. radius, tangent to the street curbs, causes the curved curb to fall within the walk intersection, a center of

FIG. 46.—Showing conventional intersection design for a business street.

FIG. 47.—Showing intersection pavement warped up to the top of the curb at the cross-walk.

curvature may be selected that will bring the curved portion of the curb to the outer corner of the sidewalk intersection but in consequence not tangent to the street curbs. The alternative is to use a shorter radius of curvature which will bring the curved curb to the outer corner of the sidewalk intersection and at the same time keep it tangent to the street curb.

At alleys and private driveways, the radius is fixed at about 20 ft. with the center of curvature at the property edge of the walk, even though the curve thus described does not come tangent to the street line. But if the distance from the center of curvature thus selected to the curb is more than 20 ft., a new center of curvature is selected to bring the curved section of curb tangent to the street curb. A little consideration of the path of a vehicle in turning corners or turning into alleys will make clear why the curve radius is fixed in this manner. These various designs are illustrated in Fig. 48.

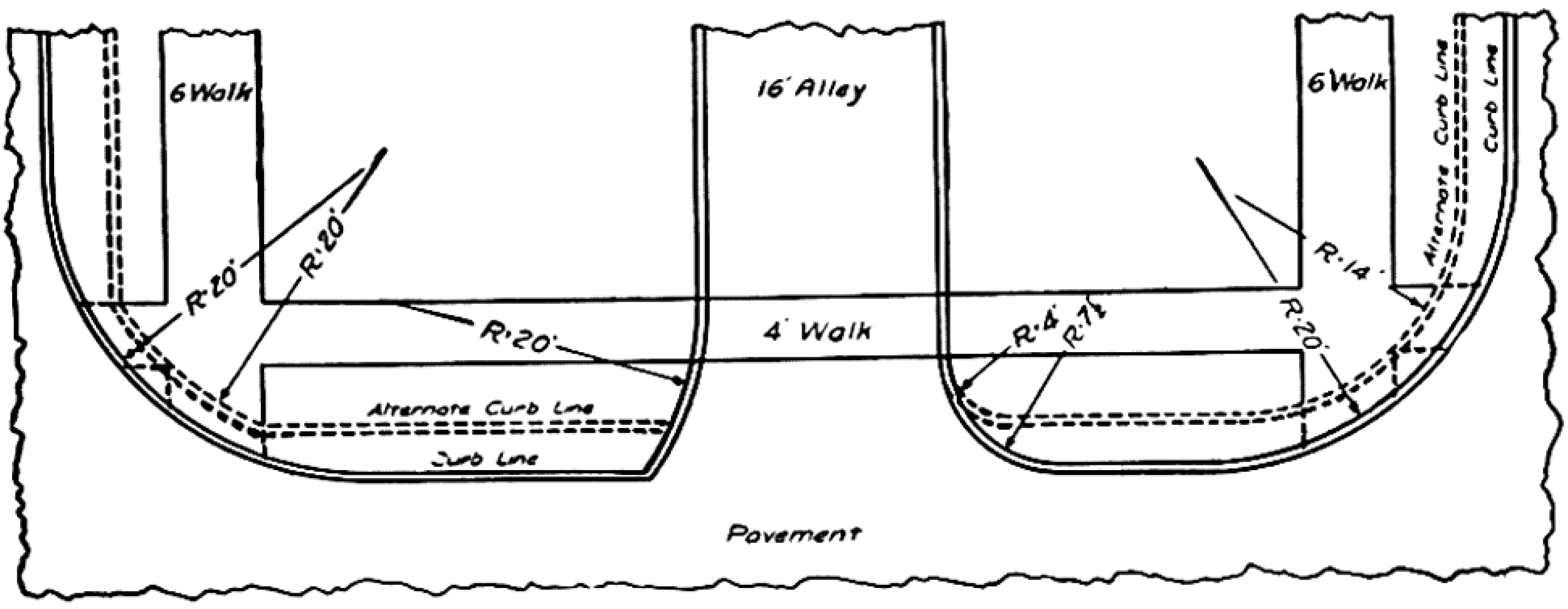


FIG. 48.—Illustrating designs of curb lines.

In districts where the entire area between the curb and the building line is occupied by sidewalks, a long-radius curb at the corner reduces the sidewalk area to an objectionable degree. Generally there is insufficient room on the sidewalk at best, and the curb radius is fixed as a compromise between two conflicting requirements. The exact radius adopted varies between 6 and 10 ft.

Elevations at Intersections.—In establishing the elevations of walks, curbs, and pavements at street intersections, due consideration is given to safety for the vehicular traffic, the comfort and convenience of pedestrians, and the disposal of storm water.

The question of safety for vehicular traffic arises more especially at intersections of streets having relatively steep grades. Adverse cross-slope to the pavement within the intersection may create a hazard for vehicles that seek to turn the corner. In the ideal intersection, which can be secured when approach grades are very light, the pavement within the intersection is either without noticeable cross-slope or else slopes downward toward each curb corner. One or the other of these conditions should be approached as nearly as possible.

The convenience of pedestrians will be served if there is no step at the curb or, if there must be one, if it is of moderate height, say 0.5 to 0.6 ft. If the pavement is warped up to the top of the curb, the slope should be flat enough to preclude serious danger when the pavement is coated with snow or ice. The sidewalks should have a moderate cross-slope, since high cross-slope is uncomfortable and dangerous. At the corner the cross-slope should be held to a maximum of 6 per cent but wherever possible should be kept down to about 2 per cent. The longitudinal slope of the sidewalk must conform to the grade of the street.

Storm water should be disposed of in such a manner as to contribute to the comfort and convenience of pedestrians and vehicular traffic, as is explained elsewhere.

The task of embodying all these diverse requirements into the design of an intersection is by no means simple, and those engineering organizations that are confronted with the problem devote a great deal of time to the solution of each specific case. Unfortunately no very definite rules of procedure can be laid down because the conditions that arise are so diverse. A few general principles (and even these are by no means universally accepted) may be suggested by way of affording a beginning of the solution of any specific problem.

SURFACE DRAINAGE

In those portions of a city that are fully paved the storm water from the paved area, the sidewalks, and all or part of that from the buildings and lawns reaches the storm sewers after traveling some distance on the pavements and in gutters that flank them. Where the area is built up with business buildings, the roof water may be carried to the street gutters by concealed pipes or directly to the storm-sewer system. But in such districts the entire street is generally covered with the pavements and the sidewalks, and all the run-off from this area follows the pavement gutters to the inlets to the storm sewers.

It sometimes becomes necessary to pave certain streets in the smaller cities or in the suburban districts of the larger ones before complete storm-sewer systems are provided. The disposal of storm water from the paved area becomes a troublesome problem in many projects of this character because it must be accomplished entirely by means of surface drains, with culverts

under streets that cross the drainage lines. If surface water must be carried across a pavement, a depression, often referred to as a "valley gutter" because of its cross-section, is provided in the surface to accommodate and concentrate the flow. But a gutter of this type is very objectionable from the traffic standpoint unless it is carefully designed with a view to its riding qualities. These cross gutters should be avoided so far as possible, but sometimes they are the only means to the end. In general, streets should not be paved until after storm sewers are installed.

Drainage Principles.—The following may be set down as the fundamental considerations of the surface-drainage system on paved streets:

1. The storm water should be disposed of in such a manner that pedestrians will not be compelled to cross streams of water that are of sufficient volume to be difficult to cross without wading. At congested crossings the cross gutter should not be in the form of an open trough or otherwise create a hazard to pedestrians. The drainage from small areas may be carried across the cross-walks in shallow depressions, but the main drainage flow should be intercepted in such a manner that it does not flow over pedestrian crossings.
2. Storm water should be disposed of in such a manner that vehicular traffic is not compelled to cross streams of surface water near pedestrian crossings and preferably not at any place.
3. The device for directing and receiving storm water should not restrict the normal traffic area of the pavement or introduce objectionable unevenness in the pavement surface.
4. The drainage of the pavement should be accomplished in a manner that precludes the collection of objectionably large volumes of water in the gutters or on the pavement surface at intersections.
5. Central gutters are preferable in alleys because there are no curbs at the edge of the alley pavement.
6. On streets that must carry large volumes of water during periods of excessive rainfall, the entire paved area may be utilized for storm water to the exclusion of vehicular traffic. Where the outlet for such water is a flume in the prolongation of the pavement, the pavement cross-section may be concave instead of convex, to increase the capacity.

Pedestrian Crossings.—At each street intersection and sometimes between intersections, provision is made for pedestrians to cross the pavement, generally in the area formed by projecting the line of the edges of the sidewalks across the intersection. This space should be kept free from streams of flowing water, if possible. The drainage water from the walk itself will of

necessity be flowing during rain storms, but most of the flow from adjacent areas can be disposed of without its crossing the cross-walks.

Storm water that is flowing toward the intersection can be intercepted before it reaches the pedestrian crossing by means of one or more inlets placed 10 ft. or more away from the cross-

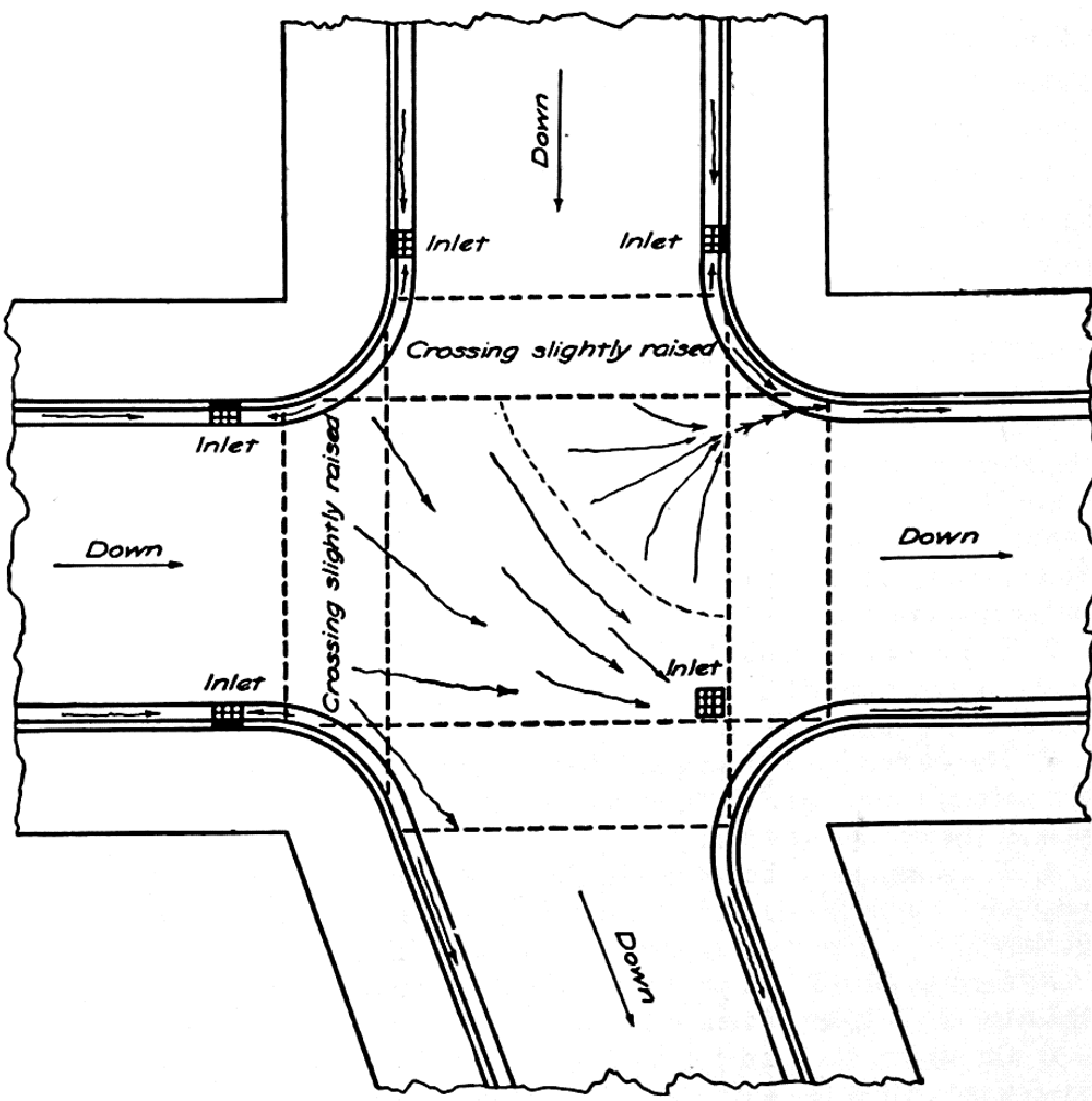


FIG. 49.—Illustrating drainage of intersection pavement.

ing. If the grades are not too steep, the storm water from the crossing itself can be made to flow to that inlet.

The storm water from the intersection area may be carried across the several pedestrian crossings in shallow depressions in the pavement without undue annoyance to pedestrians. If the intersection is of large area it may be drained to inlets near the curbs and thus disposed of before reaching the pedestrian crossings, requiring four inlets in the intersection.

Various arrangements of intersection drains are shown in Figs. 49 and 50.

Cross-drains.—At street intersections it is sometimes necessary to carry the gutter water of one street across the other. This is sometimes accomplished by continuing the gutter across the intersecting street as an open valley gutter, as has been

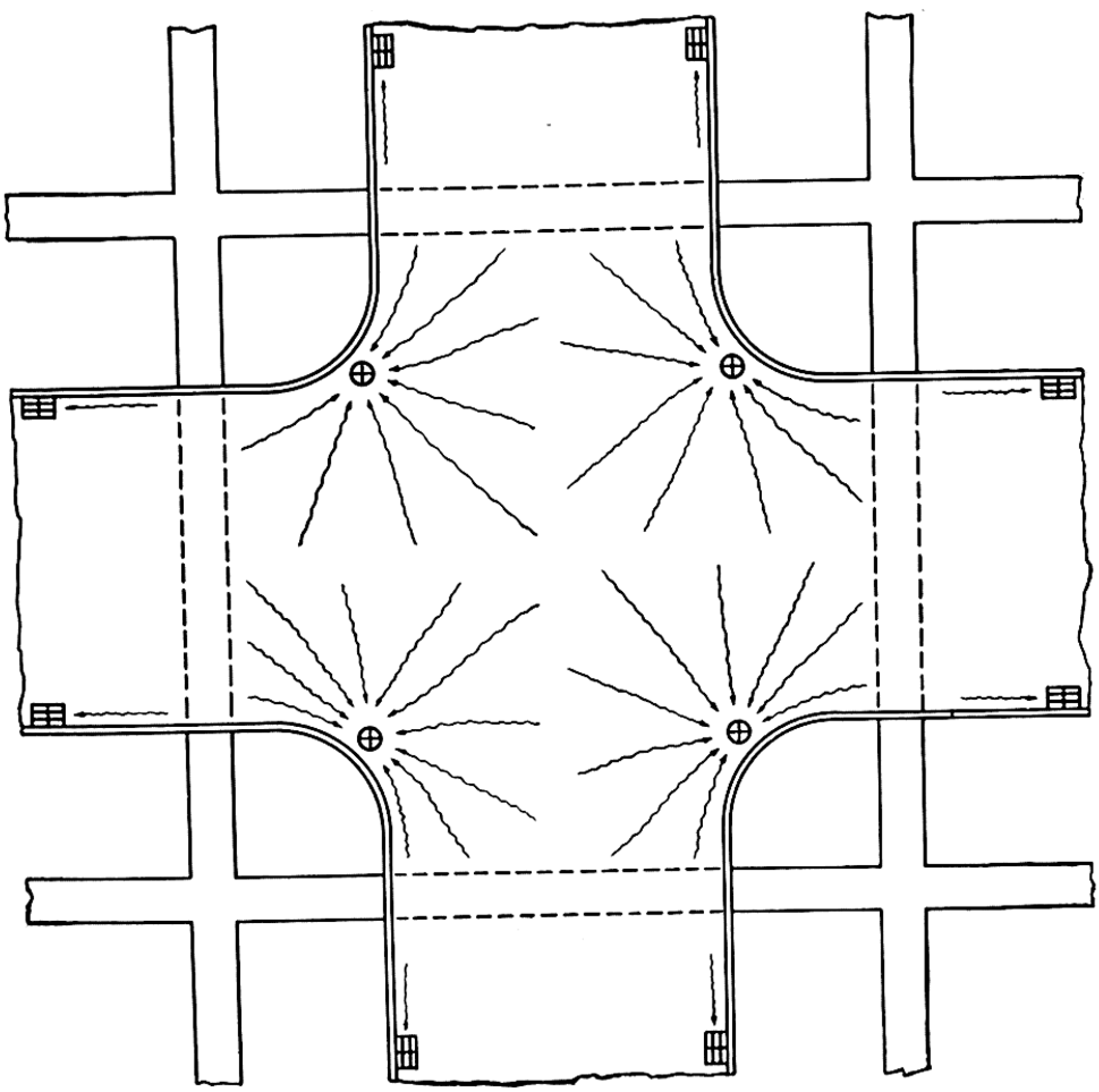


Fig. 50.—Illustrating drainage of intersection pavement.

explained. This is not objectionable if the volume of vehicular traffic that must cross the gutter is very small. It is a highly undesirable feature on streets that carry considerable traffic, even though the valley gutter may be carefully constructed with a view to minimizing the discomfort of vehicular traffic. The valley gutter may be eliminated by constructing an underdrain connected at each end to an ordinary curb inlet, as shown in Fig. 51. The storm water will enter at one inlet and flow out at the

other. That which remains in the inlets will gradually percolate into the soil, since the bottom of the inlet is not paved.[1]

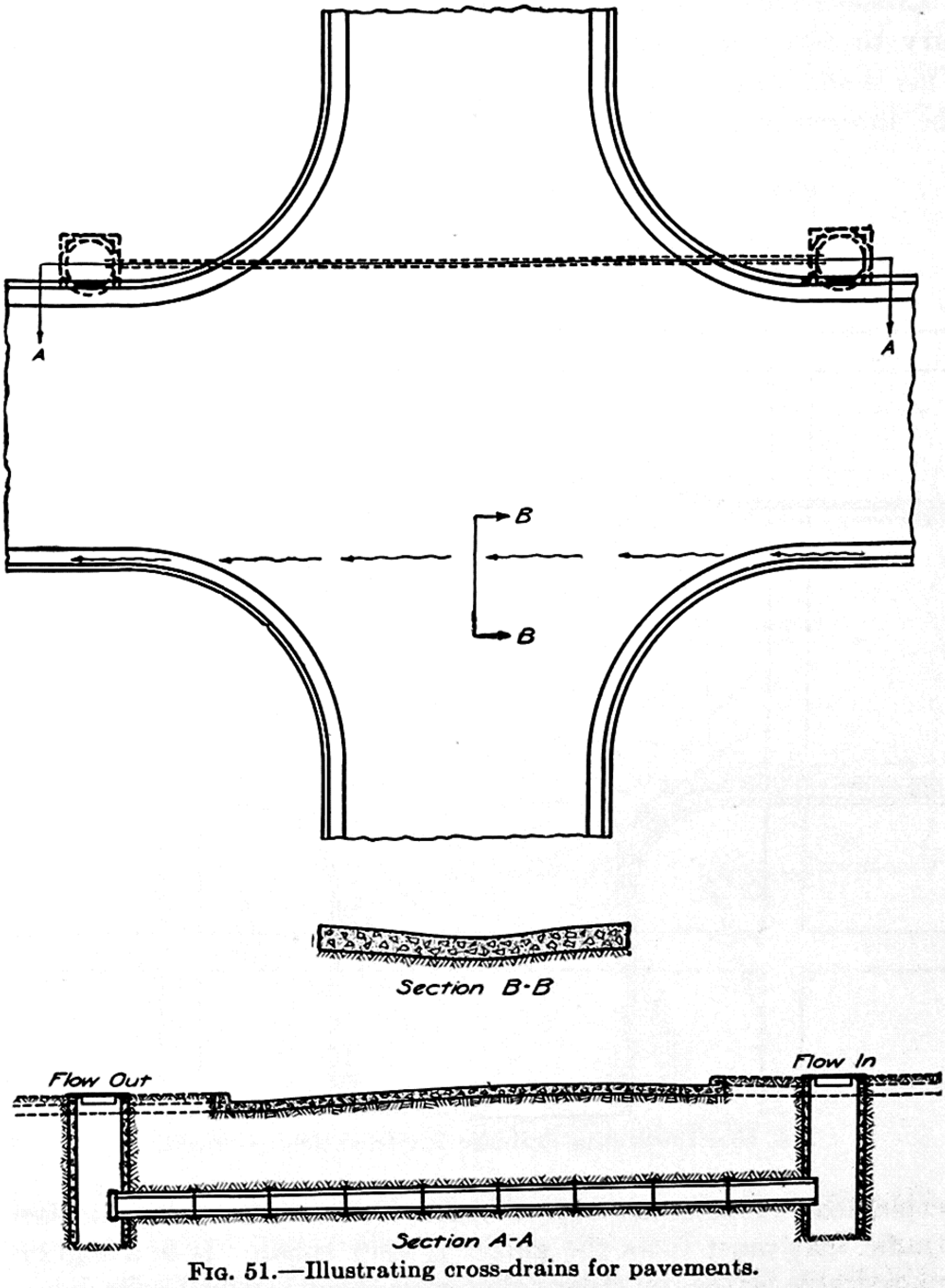


FIG. 51.—Illustrating cross-drains for pavements.

STORM-WATER INLETS

It has been pointed out that the gutters along a pavement serve to carry storm water to the inlets to the underdrainage

[1] This type of drain, as constructed in Beloit, Wis., is described in *Concrete Highway Mag.*, Vol. 9, No. 11, p. 264, November, 1925.

system. The inlets employed for this purpose are usually developed by a city to fit the ideas of the various designers who work on the problem. Many foundries market a line of castings for use as covers for inlets and manholes, and of course these are of a great variety of sizes and shapes. There is an opportunity to simplify practice in this field.

In principle there are but three types of inlets: the curb type, the gutter type, and the combined curb-and-gutter type. In each type, provision is made for removing the inlet grating or the cover to the inlet to gain access to the pipe connecting to the sewer for the purpose of cleaning. In general, the inlet drain pipe should lead to a manhole through which the storm sewer flows.

Gutter Inlets.—Gutter inlets consist of a heavy iron grating supported by an iron frame, the whole assembly resting on a special vault or receptacle from which a line of sewer pipe leads to a manhole. In some types the receptacle consists of a casting which is set in place and then surrounded with concrete. In others the receptacle consists of a brick or concrete chamber which is just deep enough to permit installing the sewer pipe that leads to the manhole. The distinguishing characteristic of this type of inlet is that the grating is designed to be placed in the pavement in the gutter area and to fit the contour of the pavement so that traffic can pass over the inlet. The gutter inlet is easily obstructed by debris carried to it by storm water, and therefore this type is not generally used on streets where the storm water carries any quantity of leaves, grass, paper, or similar refuse. It is used in those districts in the cities where the streets are kept clean by patrols and there are no trees or lawns. Typical gutter inlets are shown in Figs. 52 and 53.

Curb Inlets.—The receptacle of a curb inlet is placed behind the curb and consists of a brick or concrete chamber to receive the storm water, which flows in through an opening in the curb. The sewer pipe leading to the manhole is built in near the bottom of the receptacle. The feature of this type of inlet is that water enters by horizontal flow through an opening in the curb and consequently the opening is almost completely self-cleaning. Excessive quantities of debris may build up an obstruction that will finally close the inlet opening, but this does not occur nearly so frequently as with the gutter type of inlet. The opening in the curb may be formed in the concrete, or it may be formed by a

casting that fits the curb cross-section and affords the required opening. The larger openings are usually framed in metal. The curb type of inlet is very generally used where large quantities of water must be handled. The opening in the curb may be made of any length necessary, and the inlet receptacle of very large capacity. The opening through the curb should never be

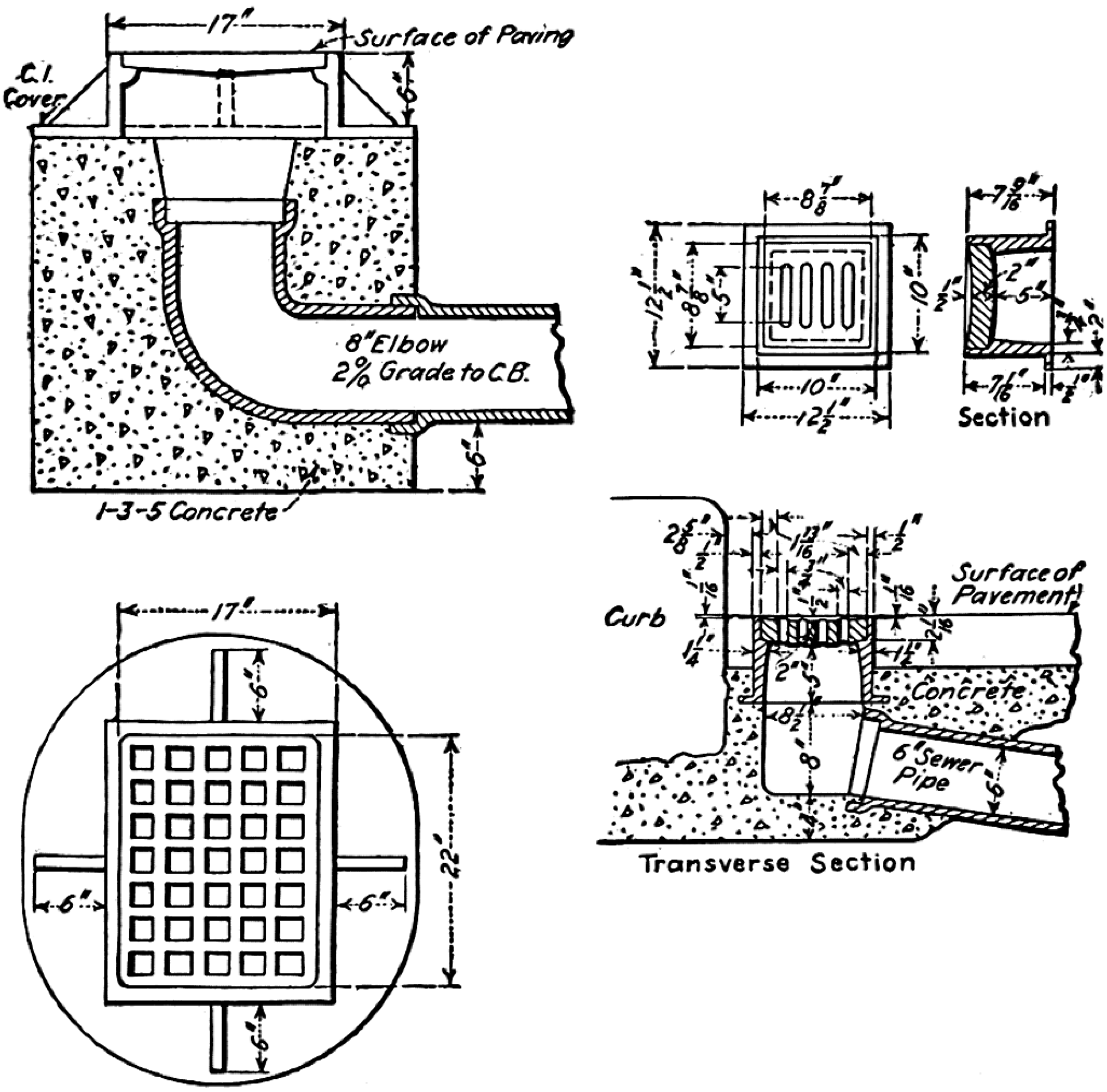


FIG. 52.—Typical curb inlet designs.

less than about 2 ft. long or less than 4 in. high and should not be fitted with a grating or vertical or horizontal bars, as these merely serve to catch floating rubbish which will clog the opening. In the older installations vertical bars were used to prevent draft animals from slipping a leg into the opening.

Curb inlets are suitable for any location where there is space behind the curb for the receptacle. It cannot be used in a city where the sidewalk fills the space between building line and pavement and the space under the sidewalk is utilized for

basement rooms. Typical curb inlets are shown in Figs. 54 and 55.

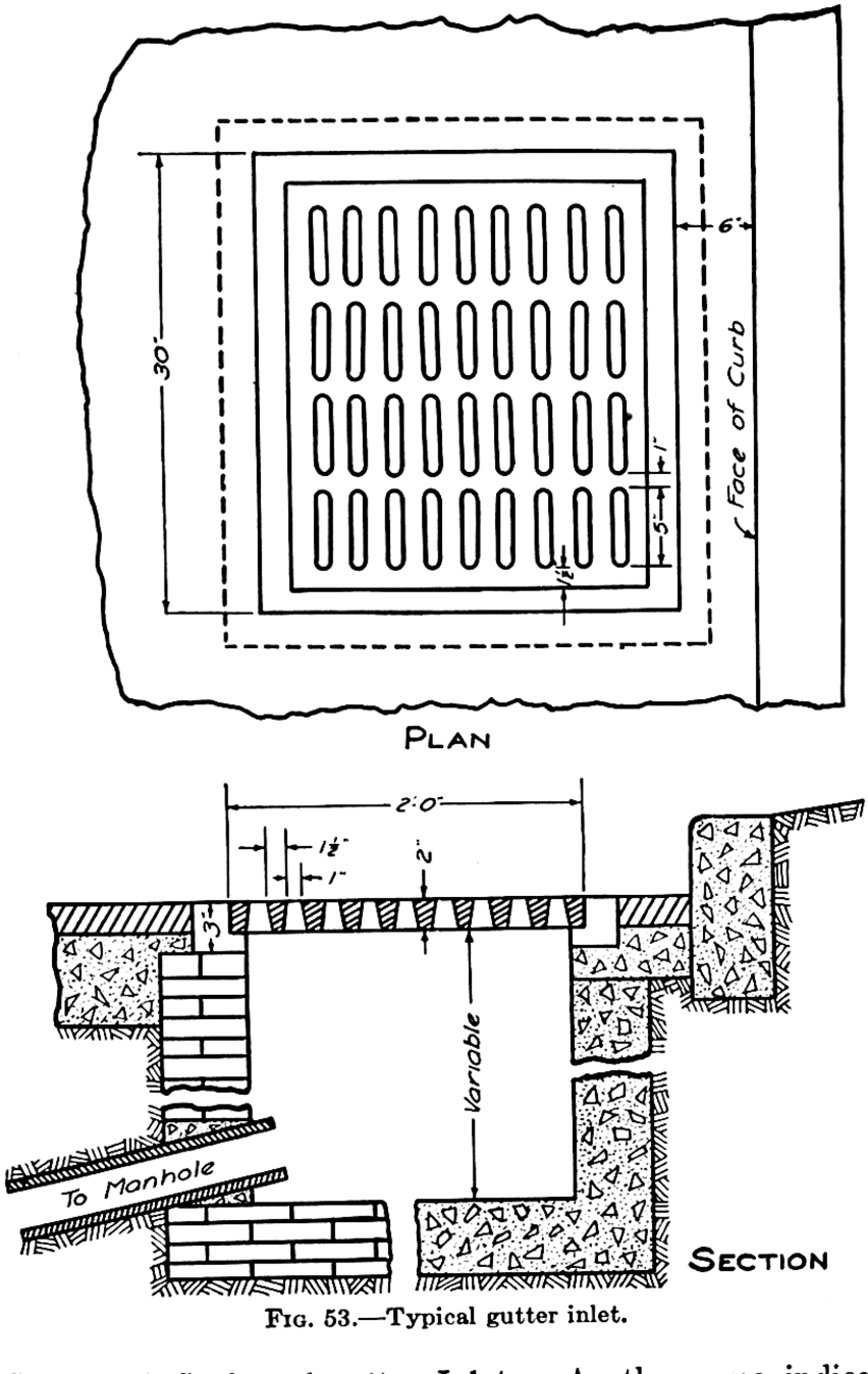


FIG. 53.—Typical gutter inlet.

Combined Curb-and-gutter Inlets.—As the name indicates, these are a combination of a gutter inlet with a curb inlet, with a single receptacle underneath. They were at one time used quite generally with combined concrete curb and gutter but are

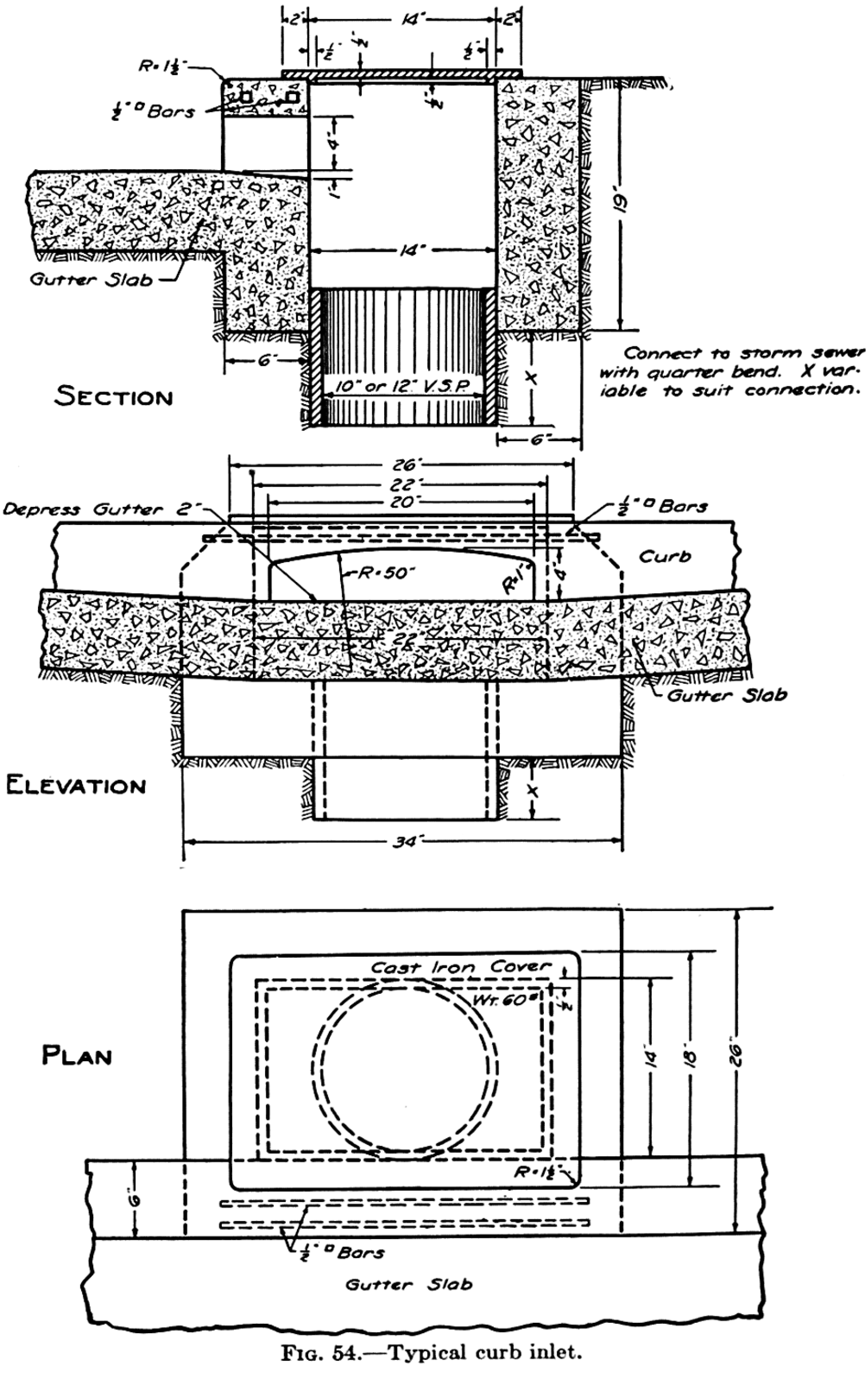


FIG. 54.—Typical curb inlet.

gradually waning in popularity for the very good reason that they cost more than the other types and are no better. Typical curb-and-gutter inlets are shown in Fig. 55.

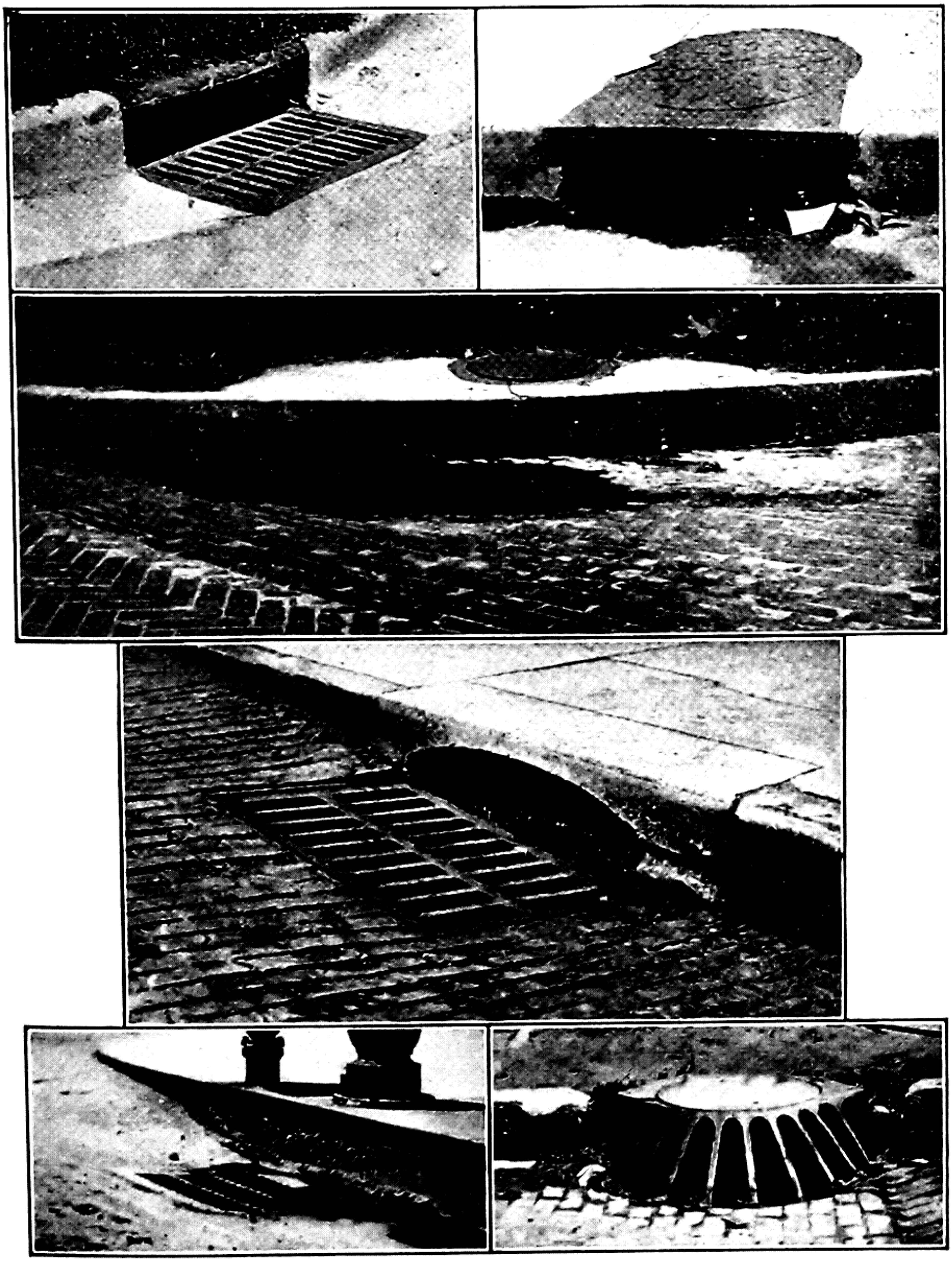


FIG. 55.—Types of inlets and castings.

Catch-basins.—A catch-basin is like an inlet except that the receptacle is considerably deeper and the outlet tile connects at some distance above the bottom of the receptacle. It was

expected that debris of various kinds would collect in the bottom of the catch-basin and not flow to the storm sewer. Of course the catch-basin has to be cleaned periodically, or it will fill up

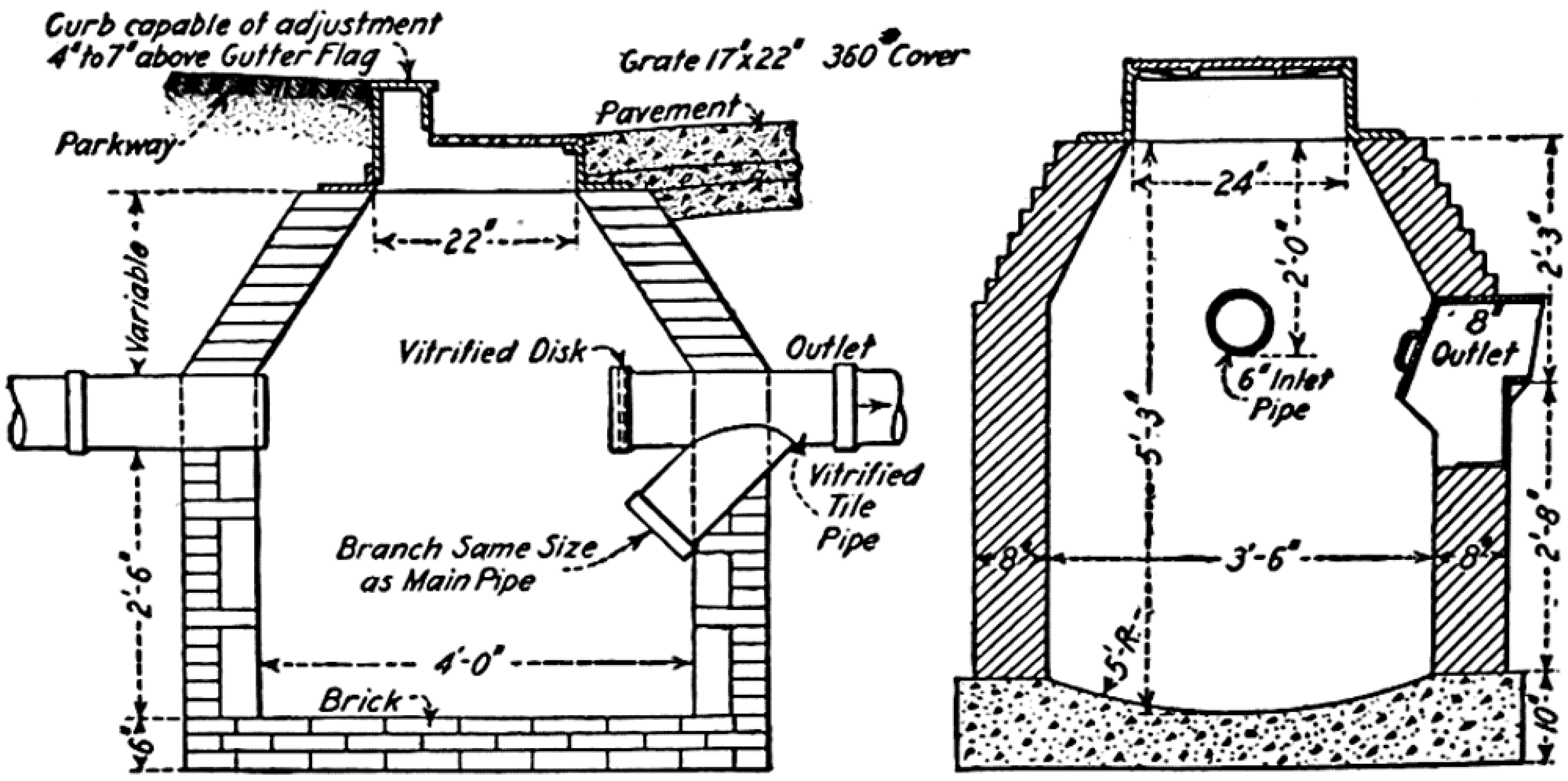


FIG. 56.—Catch-basins without trap.

until it functions just as an inlet does. The catch-basin is really a survival of the time when combined storm and sanitary sewers were used. The catch-basin could be constructed with a water trap, or seal, so that odors from the sewer could not reach the

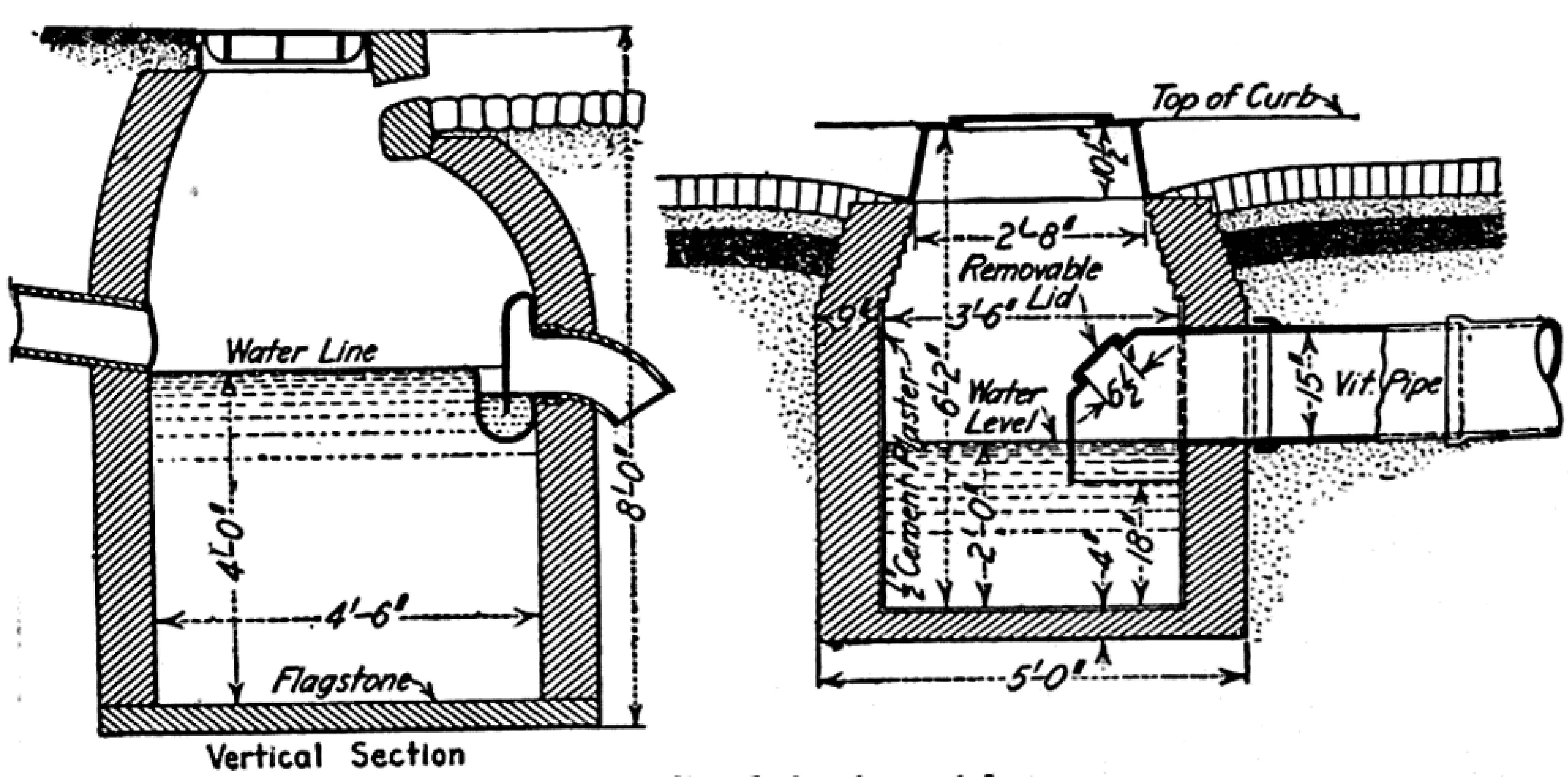


FIG. 57.—Catch-basins with trap.

street through it. Catch-basins are no longer used to any very great extent on new construction, but many of them exist on the older installations. Typical designs of catch-basins are shown in Figs. 56 and 57, and a typical manhole design in Fig. 58.

CURBS

Curbs at the margin of the pavement serve to define the edge of the traffic way, to hold in place the turf between pavement and sidewalk, to form one side of the drainage channel or gutter, and to trim and make sightly the whole street area. To a very minor degree the curb serves to retain the vehicular traffic to its allotted space, but most self-propelled vehicles with rubber tires

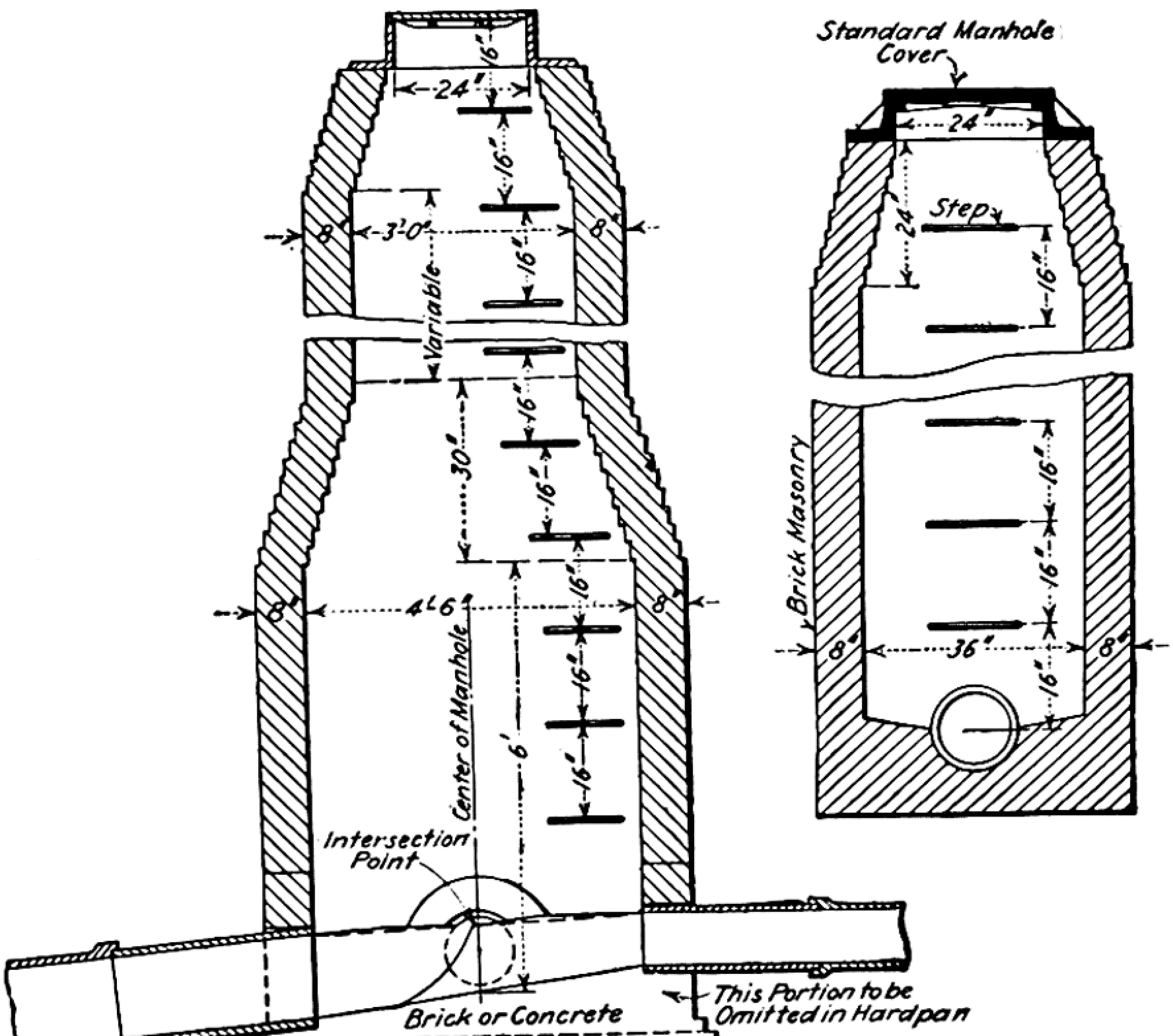


FIG. 58.—Typical manhole designs.

can readily cross any curb that would ordinarily be constructed along a pavement.

Curbs are of stone or of portland-cement concrete, and the latter type is often constructed with a gutter slab integral therewith in the form known as the combined curb and gutter. Typical curb and curb-and-gutter designs are shown in Figs. 59 and 60.

Dimensions.—The maximum height of curb is fixed by appearance and economy as well as by the desire to make it possible for a pedestrian to step readily from the gutter on to the curb.

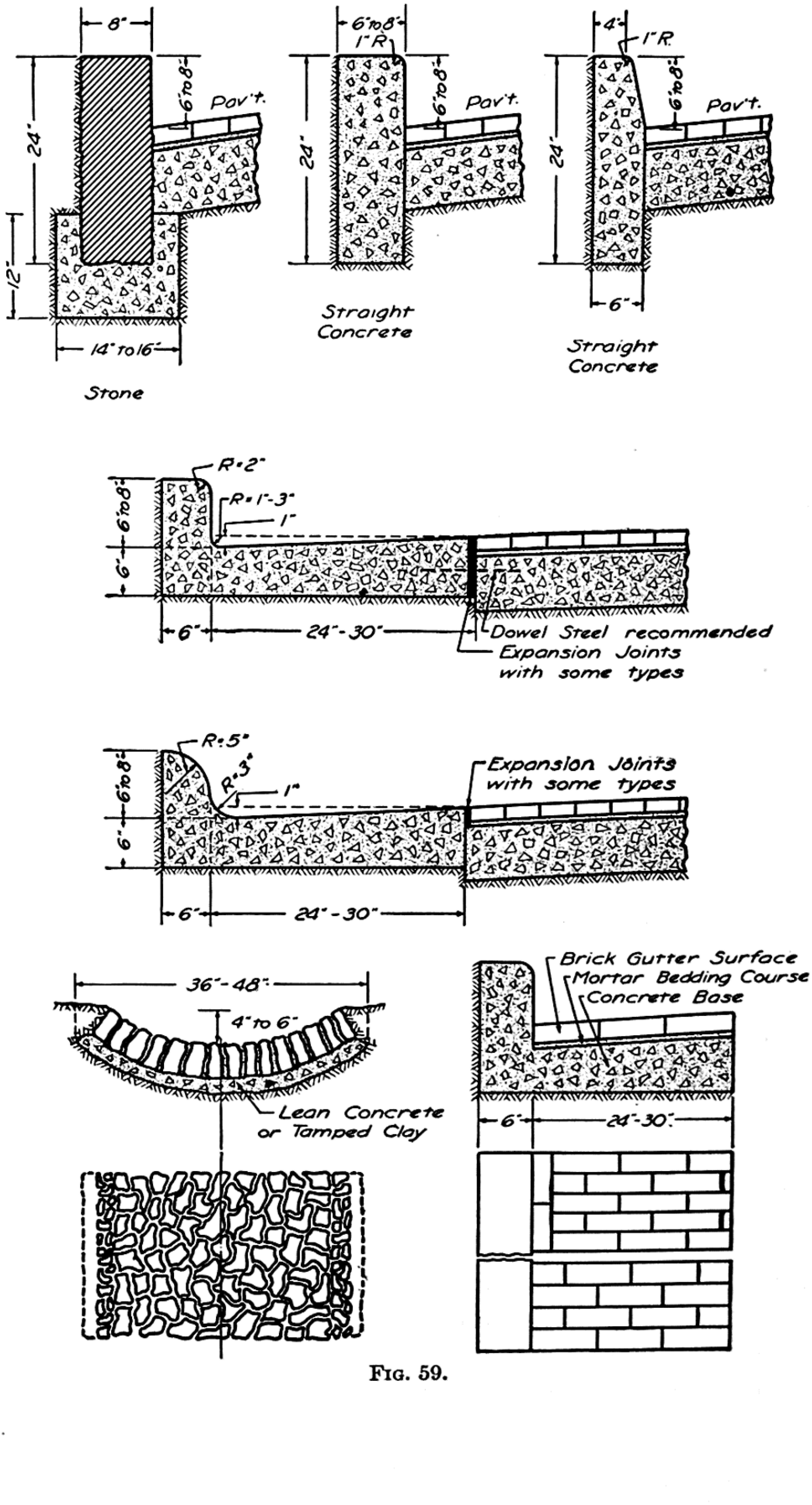

8"
24"
12"
6" to 8"
Pav't.
14" to 16"
Stone
6" to 8"
1" R
24"
6" to 8"
Pav't.
Straight
Concrete
4"
1" R.
24"
6" to 8"
Pav't.
6"
Straight
Concrete
R=2"
R=1"-3"
1"
6" to 8"
6"
6"
24"-30"
Dowel Steel recommended
Expansion Joints
with some types
R=5"
R=3"
1"
6" to 8"
6"
Expansion Joints
with some types
6"
24"-30"
36"-48"
4" to 6"
Lean Concrete
or Tamped Clay
Brick Gutter Surface
Mortar Bedding Course
Concrete Base
6"
24"-30"

Fig. 59.

The requirement is not very rigid, but 0.6 ft. is generally considered to be the maximum height desirable, with 0.5 ft. the

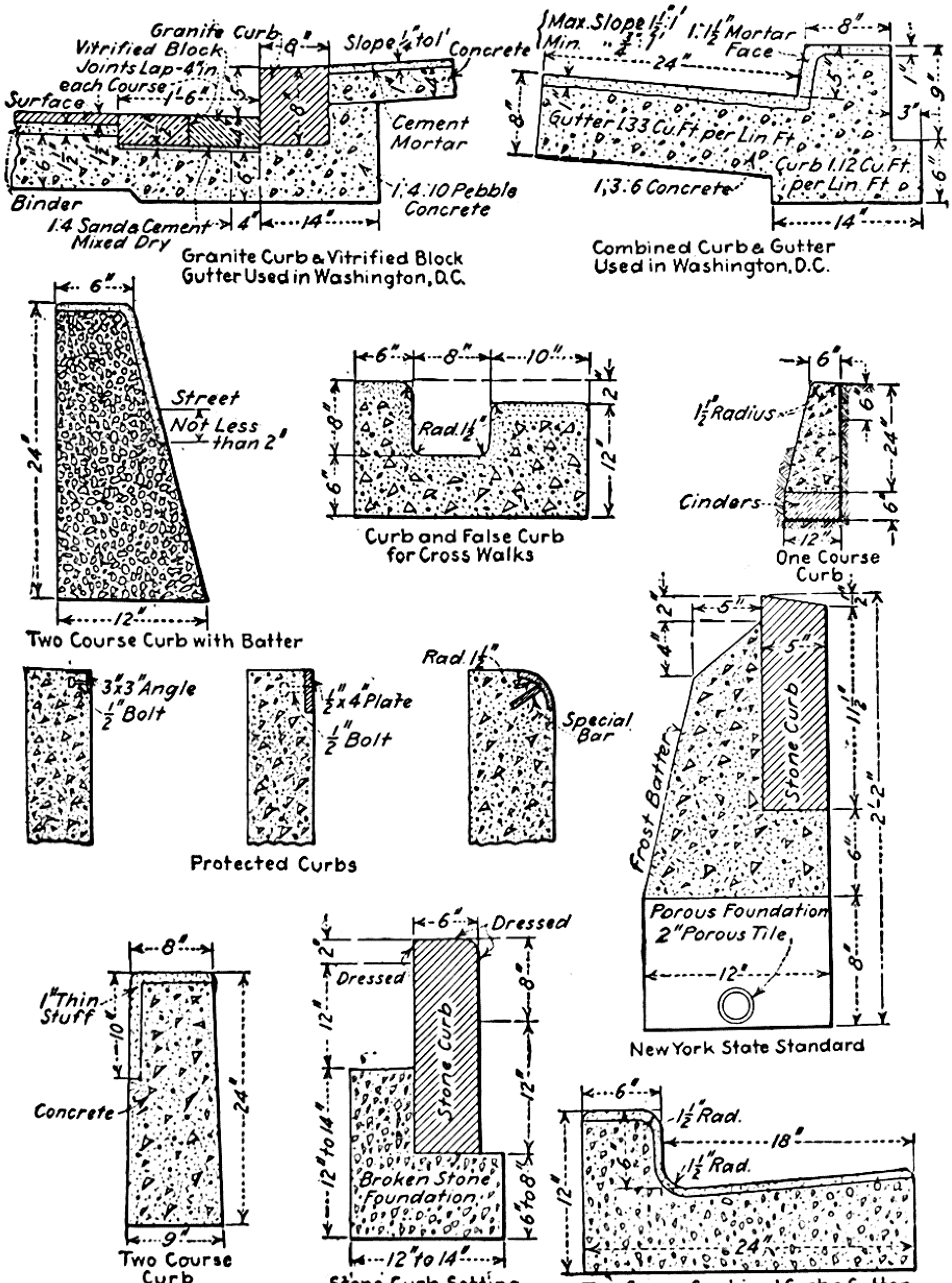


Fig. 60.—Designs of curbs and combined curb and gutter.

most common height. As might be expected, considerable latitude is allowed the designer with respect to curb height.

The thickness of the curb is generally about 6 in. except where heavy loads are likely to bump the curb, when the thickness is

generally 8 in. or more, and in these locations the granite curb is more likely to be used than any other type.

Foundations.—Stone curbs are generally set on concrete, which is carried up both the front and back faces of the curb to hold it securely to line and grade. Where drainage conditions are bad and the curb is likely to be shifted owing to the freezing of the water in the soil under the concrete, a layer of porous material such as gravel, broken stone, cinders, or broken slag is placed under the concrete foundation. Where very stable soils exist and the climate precludes there being any disturbance due to freezing, the curb is bedded on a layer of tamped gravel or cinders. It will readily be apparent that in order to set stone curbs accurately and permanently to grade, some kind of stable bedding layer is necessary.

Concrete curbs are quite commonly placed directly on the natural soil, which is shaped and thoroughly tamped before concrete is placed. Curbs of this type can be constructed to line and grade. If bad drainage conditions exist, or the curb is likely to be disturbed by the freezing of the water in the soil under the curb, a bed of cinders, gravel, or other porous material is placed and tamped before the curb is constructed.

Combined concrete curb and gutter is placed on foundations exactly like those employed for concrete curbs.

Expansion and Contraction.—Curbs of all types and combined curb and gutter expand and contract from the effects of temperature and moisture; and unless adequate provision is made to compensate for this change in dimension, the curb will be ruptured by the stresses induced. This rupture is quite likely to take place on curved sections where the curbs turn on to intersection streets or alleys.

Expansion of stone curb is provided for by leaving a space of about ⅛ in. between adjacent sections. Sometimes this space fills with inert material in the course of time and becomes inoperative as an expansion joint, and it is a good plan to provide somewhat larger joints between the sections near curves.

Expansion of concrete curbs or curb and gutter is provided for by constructing them in sections about 6 ft. long with spaces of about ⅛ in. between the sections. As with stone curbs, it is advisable to provide additional space for expansion near curved sections. This is sometimes done by leaving a space of about 1 in. between the last straight section and the first curved section of a line of curb or curb and gutter.

Curbing is often pushed out of line and sometimes actually ruptured by the thrust due to the expansion of sidewalks that abut the curb or that end at the curb. This sort of damage can be prevented by allowing ample expansion joints between curb and walk. In the case of sidewalks that end at a curb, as at intersections, a very wide expansion joint is necessary because sidewalks tend to increase in length with age. The expansion joints between adjacent slabs of the walk fill with inert material and cease to function as expansion joints; consequently, the

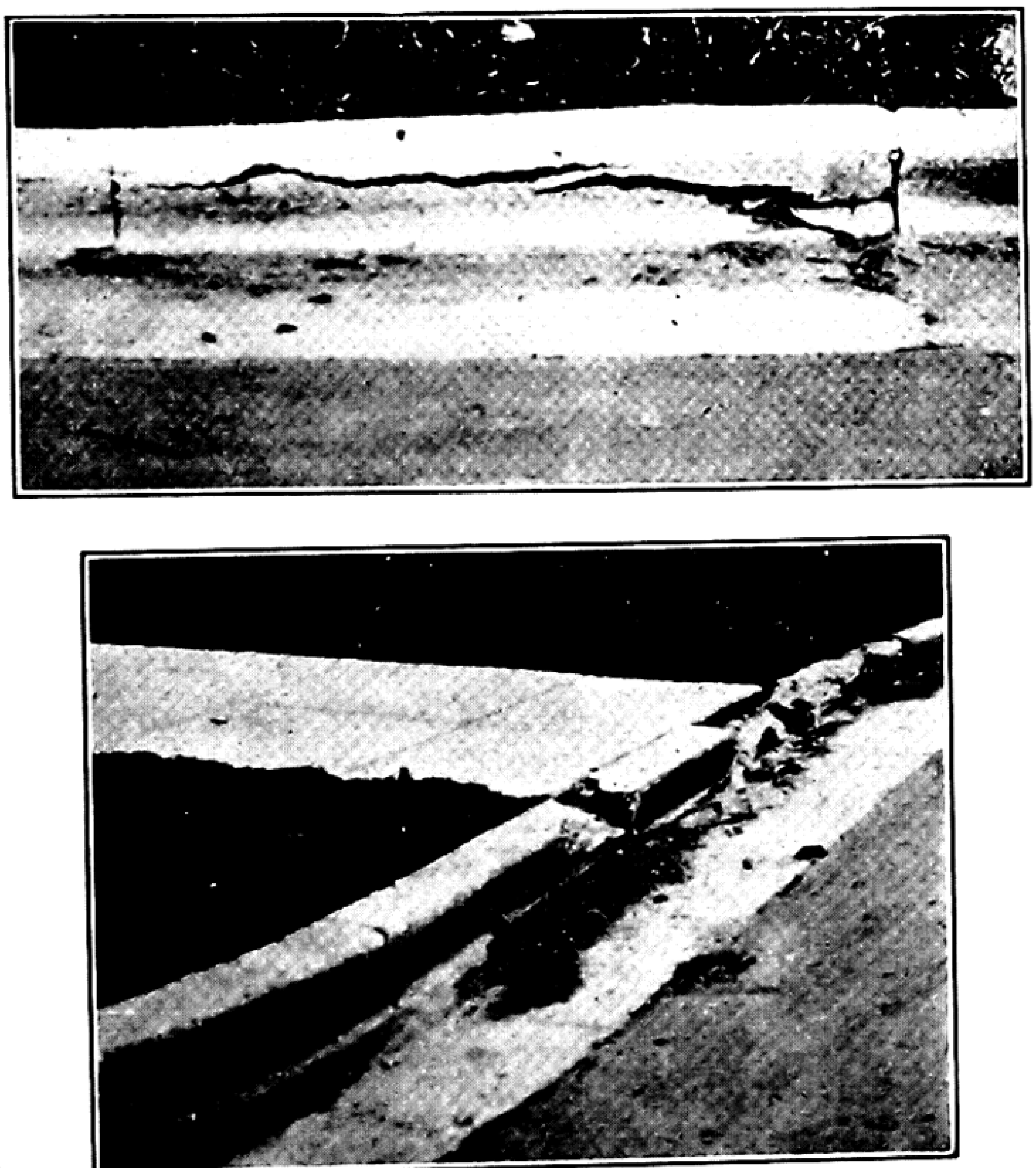


FIG. 61.—Illustrating need of expansion joints in curbs and walks.

expansion is carried the full length of the line of walk. But the walk is in short independent sections, and it does not decrease in total length when it contracts. In some cities the end of the walk is built into a notch in the curb and may therefore expand indefinitely without disturbing the curb. Instances have been noted where the end of the walk had been thrust several inches beyond the face of the curb by expansion, but with the slot design no damage to the curb resulted. The damage that results from inadequate provisions for expansion is illustrated in Fig. 61.

Ornamental Gutters.—Sometimes it is desirable to add an ornamental feature to the gutter by way of giving a pleasing effect to the street design. This is done by using red vitrified brick or cobble stones for the gutter slab. These are placed in various designs on a concrete gutter base. This design is particularly effective in combination with asphalt pavements.

In parks, estates, cemeteries, and suburban districts cobblestone gutters are very effective and are often used. Generally the gutter is constructed without a curb, and field stones of various sizes and colors are mixed indiscriminately in the gutter construction. These gutters are generally 2.5 to 3 ft. in width and about 6 in. deep. The stones are set in lean concrete or, if the soil is stable, on a carefully shaped and tamped soil foundation. Generally the spaces between the stones are filled with a lean concrete grout.

Concrete for Curbs.—For many years concrete curbs and curb and gutter were constructed of lean concrete with a fairly rich cement mortar facing about ½ in. thick on all exposed faces. The concrete for the body of the curb-and-gutter slab corresponded to a 1–3–6 mixture, and the facing was 1–2 mortar. In recent years the trend has been toward the use of one mixture throughout, generally about equivalent to 1–2½–4 concrete. This provides sufficient mortar to insure good surfaces next to the forms. After the forms are removed, the exposed surfaces may be rubbed with water, and a wood float to remove lines left by joints in the forms. If metal forms are used, it is rarely necessary to do any extensive dressing after the forms are removed. Curbs of this mixture are strong enough to resist all ordinary impact from vehicles.

Armored Curb.—Curbs adjacent to freight-receiving platforms and at other places where they are subject to abrasion from wheels of vehicles are sometimes constructed with a metal plate covering the upper 2 or 3 in. of the exposed face of the curb to reinforce it and protect it against abrasion. If the upper edge is rounded, the protection plate is shaped to the radius of the edge.

SIDEWALKS

The design of sidewalks is fairly simple and reasonably well standardized, but the actual construction of sidewalks is too often sadly neglected. For general utility walks, which com-

prise the major proportion of those constructed each year, the portland cement concrete type is employed. Some other types are used where it is desired to secure ornamental effects.

Width of Walks.—Sidewalk widths are based on traffic requirements with 2 ft. the width of a pedestrian traffic lane. Pedestrian traffic counts and the analysis thereof are of some use in estimating the required width of sidewalk and have been used to a limited extent. In the main, little opportunity exists for exactness in this matter, and walks are constructed of widths determined by judgment and experience.

In outlying residential districts the width is generally 4 or 5 ft.; in well-built-up residential districts, 6 to 8 ft.; and in business districts, 10 to 30 ft.

Cross-slope.—Sidewalks on streets are constructed to drain toward the gutter, and consequently the slope is continuous across the walk at the rate of about 1/4 in. per foot of width. At intersections it sometimes becomes necessary to use a much greater cross-slope for short distances, but this is held to a maximum of 6 per cent if possible.

Walks in parks, private grounds, cemeteries, and similar locations may be constructed with cross-slope just as ordinary street walks are, or they may be crowned slightly. If crowned, the rate of crown is fixed at about 1/4 in. per foot.

Two-course Concrete Walks.—This type of walk consists of about 4 in. of lean concrete (the equivalent of a 1–3–6 mixture of concrete with good aggregates) and 1/2 in. of wearing surface of 1–2 or 1–2½ mortar.

If drainage is poor, the walk is provided with a ballast layer of porous material such as cinders or gravel. More often, the base course is placed directly on the natural soil.

The base course of rather dry concrete is placed and tamped, and the wearing surface is then placed on the green base, struck off to the level of the forms, and trowel-finished. To get a smooth, glazed appearance a steel trowel is used. To secure a granular, non-slippery surface a wooden float is used for finishing. Sometimes the surface of the green concrete is dusted with granite chips, carborundum, or some similar sharp-grained substance, which is incorporated into the surface by floating or troweling, thereby reducing the tendency to slipperiness.

The walk is marked into sections about 4 ft. square with a grooving tool to create planes of weakness which will later develop

into cracks and minimize the formation of irregular cracks. Sometimes the walk is actually separated into sections by cutting the green concrete with a trowel and then finishing at the cut with a grooving tool.

One-course Concrete Walks.—Single-course concrete walks are composed of a single layer of concrete mixed in the proportions about equivalent to 1–2½–4 concrete of ordinary good aggregates. Otherwise, these walks are constructed in the same manner as two-course walks.

Expansion and Contraction Joints.—Contraction joints form by rupture at the planes of weakness resulting from the grooving of the concrete walk as it is finished. Expansion joints, which are merely open joints about 1 in. wide, are provided near the intersection with other walks, and at the ends of the walk that abut curbs a joint 2 or 3 in. wide is provided. When the edge of the walk is adjacent to a curb, an expansion joint ½ to 1 in. wide, depending upon the width of the walk is provided by inserting a sheet of expansion joint filler or by constructing an open joint which is later filled with bituminous material. These joints are filled because water flows across them and should be prevented from finding its way down behind the curb.

When walks are built over vaults or rooms, prism glass is installed, and the walk consists of a concrete slab designed to carry a load. It must be kept watertight. Closely spaced expansion joints are provided which are caulked with oakum and then filled with a bituminous material.

ORNAMENTAL WALKS

In estates, parks, and similar places it is sometimes desired to secure a walk that is more sightly than the ordinary portland-cement concrete walk. Several types of ornamental walks are used in such locations.

Tinted Concrete Walks.—By mixing lampblack, red ocher, or other mineral colors in the concrete for the finish coat, the walk can be tinted as may be desired. Some very good effects are secured in this way. Care must be exercised in selecting the sand to be used in such cases to make sure that the color in conjunction with the sand does not produce a blotchy surface, but any uniform sand, thoroughly washed, will be satisfactory.

Bituminous Walks.—If black is the color desired, it may be secured by applying a thin bituminous carpet to the concrete

walk according to the method described for bituminous carpets on macadam in Chap. XV, except that the dressing is generally rather fine sand and is used sparingly.

Bituminous walks of penetration macadam are also satisfactory if carefully constructed. The method is identical with that described for penetration macadam pavements in Chap. XVI, but the total thickness is rarely more than 5 in., and the top course is about 1 in. thick.

Bituminous carpets are sometimes applied to gravel or macadam walks but are not very satisfactory.

Pebble Concrete Finish.—An effective finish to concrete walks is obtained by scattering pebbles, carefully screened to pass a ½-in. screen and be retained on a ⅛-in. screen and of good colors, over the surface of the concrete as soon as it has been struck off to the correct level. The pebbles are then tamped or rolled until they are thoroughly bedded in the mortar. After the cement has begun to harden, the surface is brushed vigorously with a stiff brush broom to expose the rounded pebbles.

A similar effect is secured by using for the surface course about 1 in. of concrete made of screened pebbles. The finish course is spread and carefully tamped or rolled to the desired contour, and then the mortar is brushed out to expose the pebbles.

Tile Walks.—Tile walks are constructed by laying tile of various colors and sizes in patterns on a concrete base. The base is of concrete of about 1–3–5 proportions, and the bedding layer is of lean cement mortar.

Gravel Walks.—These consist of a 5-in. layer of bonding gravel, screened to remove all pebbles larger than ½ in. in size. Such walks must be smoothed carefully at intervals for several months and may be expected to soften somewhat when there is any considerable amount of rain; but if care is exercised to secure gravel with low clay content, this will not occur to an objectionable extent.

DESIGN OF STREET INTERSECTIONS

One of the most troublesome problems encountered in street design is the establishment of grades at intersections. So many and diverse are the conditions encountered that no simple rules can be laid down covering all cases. Moreover, practice has not been anywhere nearly standardized, nor can it be. The design of intersections is therefore a matter for special study

every time it presents itself. A few fairly well-defined principles may be laid down that will serve as a basis for the work but the details will vary greatly among designs that are acceptable.

Streets with Grades Less than 2 Per Cent.—(1) The center-line grade is carried without a break across the intersection and

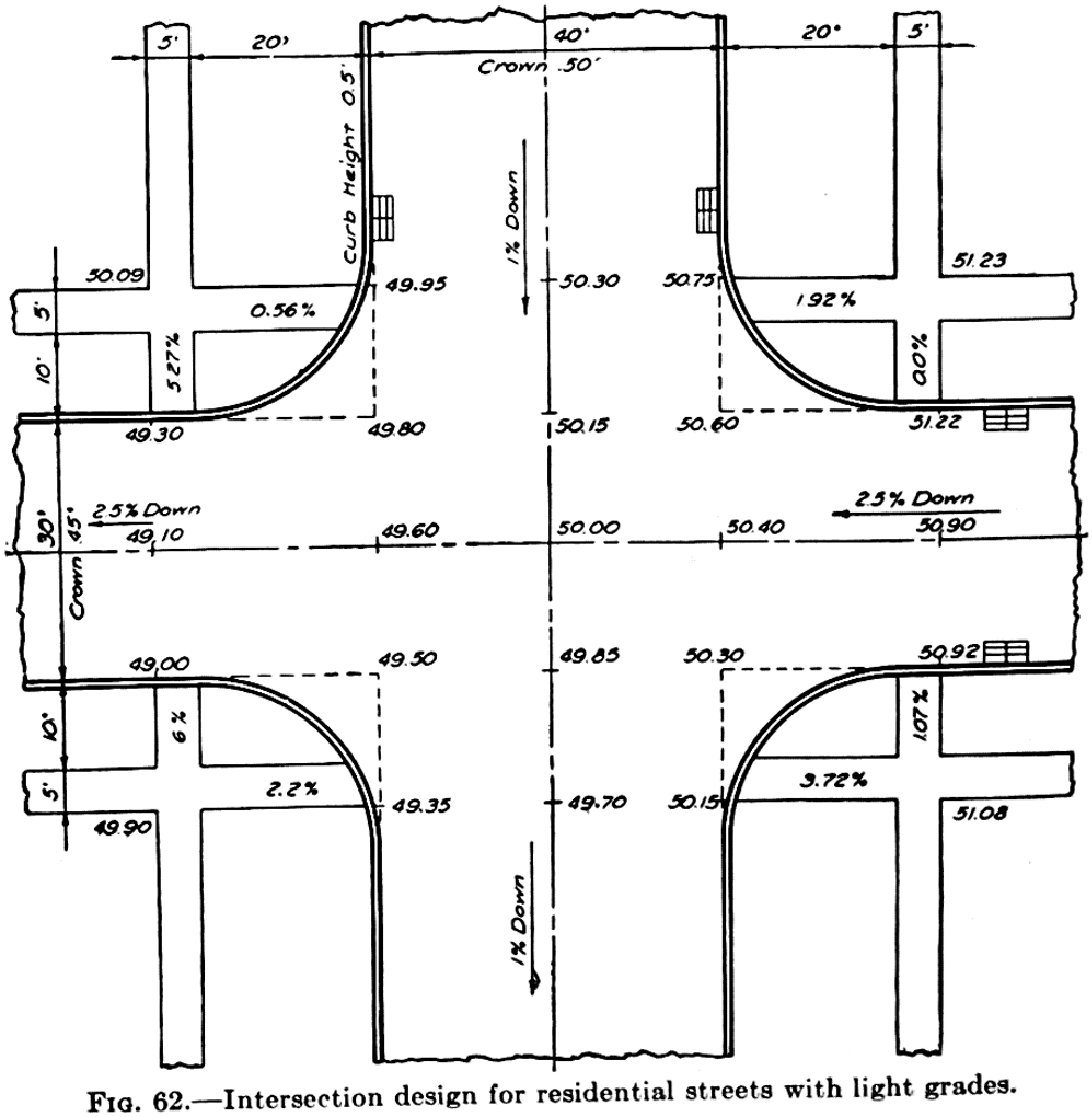


FIG. 62.—Intersection design for residential streets with light grades.

the curbs on opposite sides of the street are not at the same elevation. The difference in elevation on a 40-ft. street with a 2 per cent grade will be 0.8 ft. and this can readily be taken care of by either varying the height of the curb so as to keep the gutters at nearly the same elevation, or by using an unsymmetrical cross-section for the street, or by both means. It may be desirable to keep the curbs and the pavement cross-section symmetrical on the more important of two intersecting streets, and have a break in the grade line of the other at the curb line of

the first. This is frequently done where one street is a main thoroughfare and the other a cross street.

As a general rule difficulty is experienced in securing a safe and properly drained intersection if the difference in elevation of the opposite curbs is much greater than 1 ft., *i.e.*, on a 50-ft. pavement when the grade of the intersecting street is 2 per cent.

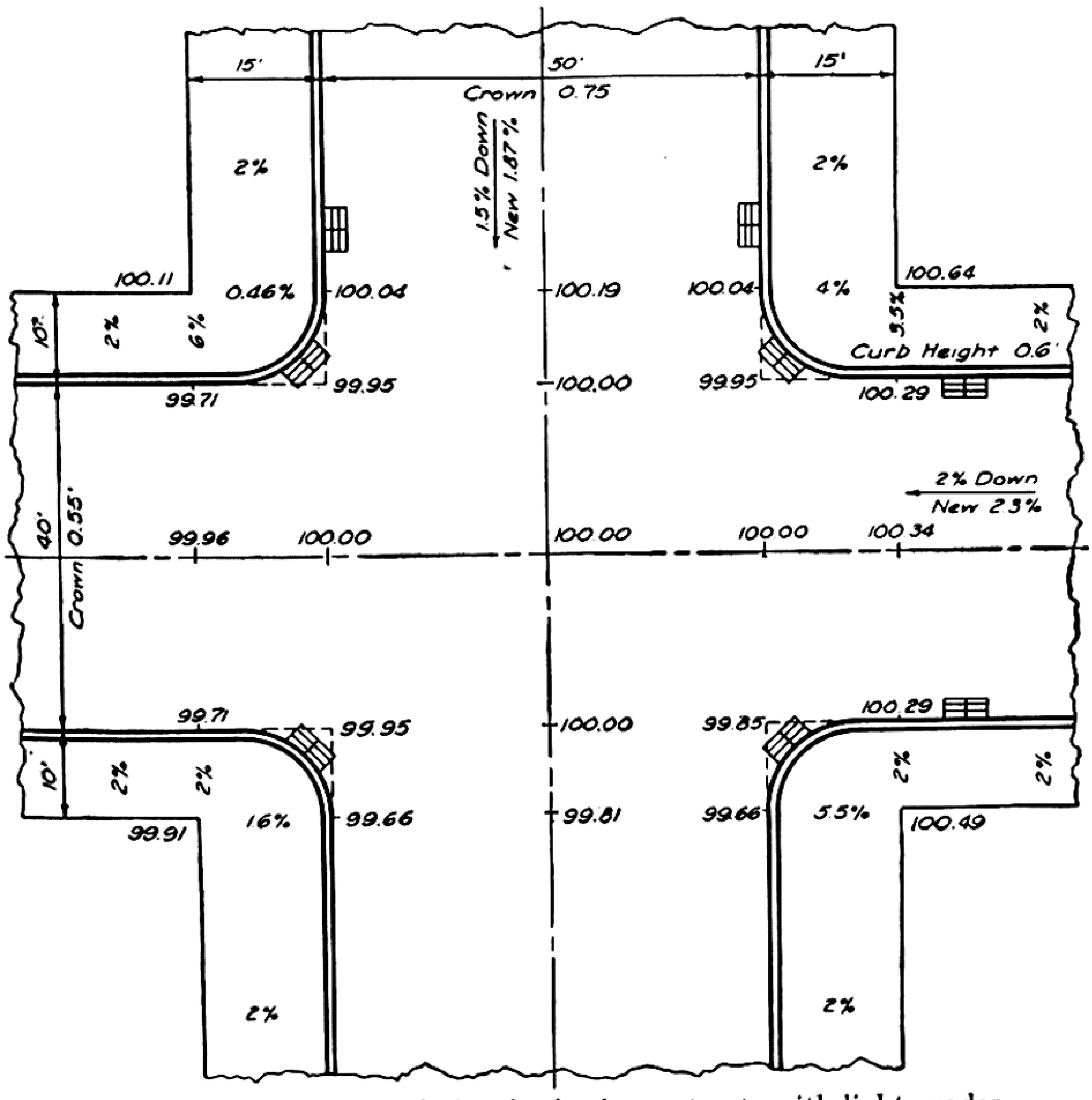


FIG. 63.—Intersection design for business streets with light grades.

When such is the case, method (2) may be used. Figure 62 shows the design of an intersection for a residence street with light grades.

(2) The other method of designing intersections for streets of light grades is to make the elevations of the four curb corners the same. The grade lines of each street will then break at the curb lines of the intersection. This design is generally easily carried out on residence streets, but may be difficult in a business district on account of the established building-line elevations.

From the standpoint of securing an intersection that is safe for vehicles it is to be commended. It has the disadvantage of putting the curb line somewhat below the level of the lots on the uphill side of the street, but this is a secondary consideration. Figure 63 shows the design of an intersection for a business street with light grades.

Streets with Grades over 2 Per Cent.—If streets are on grades greater than 2 per cent, and especially if they reach 5 per cent or more, it is generally impossible to carry the grades continuously across intersections because of the difference in height of curb that would result, although it might be done if the grades lay between 2 and 4 per cent. The grade cannot always be broken at the curb line because of the difficulty of securing reasonable grades to the sidewalks, especially on business streets. On residence streets it is sometimes possible to adjust the grades of walks if the street grades do not exceed about 4 per cent and make the break in street grade at the curb line of the intersecting street. When that is possible it is the best solution of the problem, as it usually increases the grades on the remainder of the block less than any other method of treatment. Figures 64 and 65 show designs of intersections for grades in excess of 2 per cent.

For business streets, satisfactory grades at the intersection are secured by flattening the street grade across the intersection, making the break in the grade line at the property line of the intersecting street. It will easily be seen that this increases the grade along the remainder of the block.

This emphasizes the desirability of flattening the grade across the intersection only so much as is necessary to secure a satisfactory intersection. This can be determined only by repeated trial for any given intersection. On account of the variation of width of pavement and sidewalk no uniformity exists in the procedure; but whatever modification is made looks to one end, namely, to secure an intersection with reasonable slope and sidewalks that fit to the curb grade without undue cross-slope. This cross-slope of the walk may be as great as 6 per cent and in exceptional cases 10 per cent but for ordinary cases should be held as low as 2 per cent.

For the intersection of streets crossing at excessive grades or for streets intersecting at acute angles the problem becomes very complicated, and the procedure outlined by Vernon S. Moon

in a paper read before the Municipal Engineers of the City of New York has been widely quoted and commended. It is given at the end of this chapter as one of the examples of good practice. Besson has employed a system of contours for use in studying grades at street intersections and has used the method with good results, in designing difficult intersections.[1]

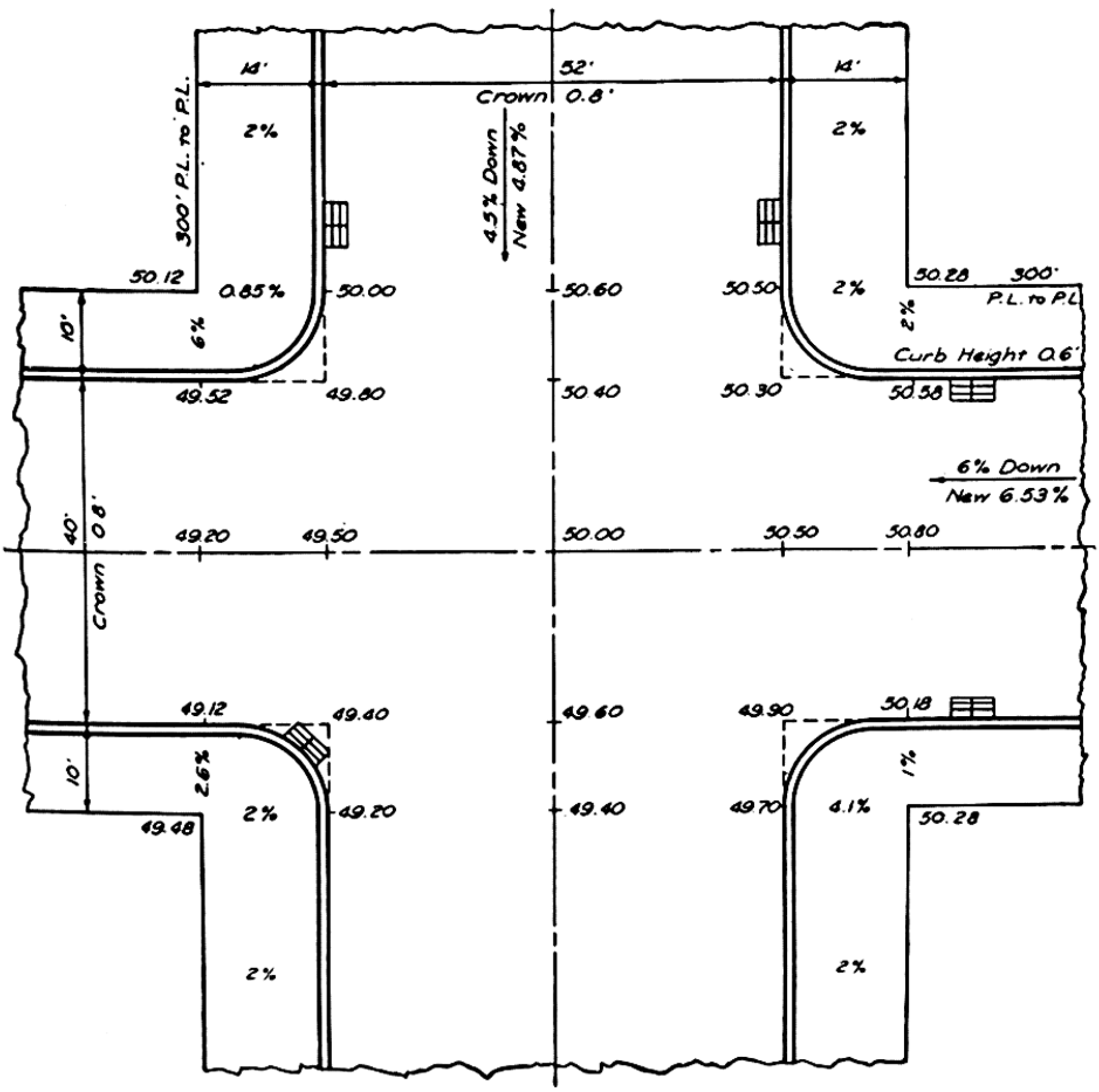


FIG. 64.—Intersection design for business streets with moderately heavy grades.

CAR-TRACK PAVING

Whether car-track paving shall be of the same material as that used on the remainder of the street depends upon several conditions. Any street that carries a large amount of heavy truck traffic must have a very durable type of pavement, and the car-track paving may be of the same type as the remainder of the street. Streets that are paved with stone blocks, wood

[1] BESSON, F. S., "Street Intersections Mapped Out by Means of Contours," *Eng. News-Record*, Vol. 90, No. 6, p. 255, February, 1923.

blocks, or vitrified brick generally have car tracks paved with the same material.

If the traffic is mixed and is medium or light in volume, the car-track paving may be of more durable material than is used for the remainder of the street. At crossings where the tracks curve around corners or the trackwork is otherwise complicated, the paving is likely to be cut up into small areas which receive severe service. In such cases it is good practice to provide a

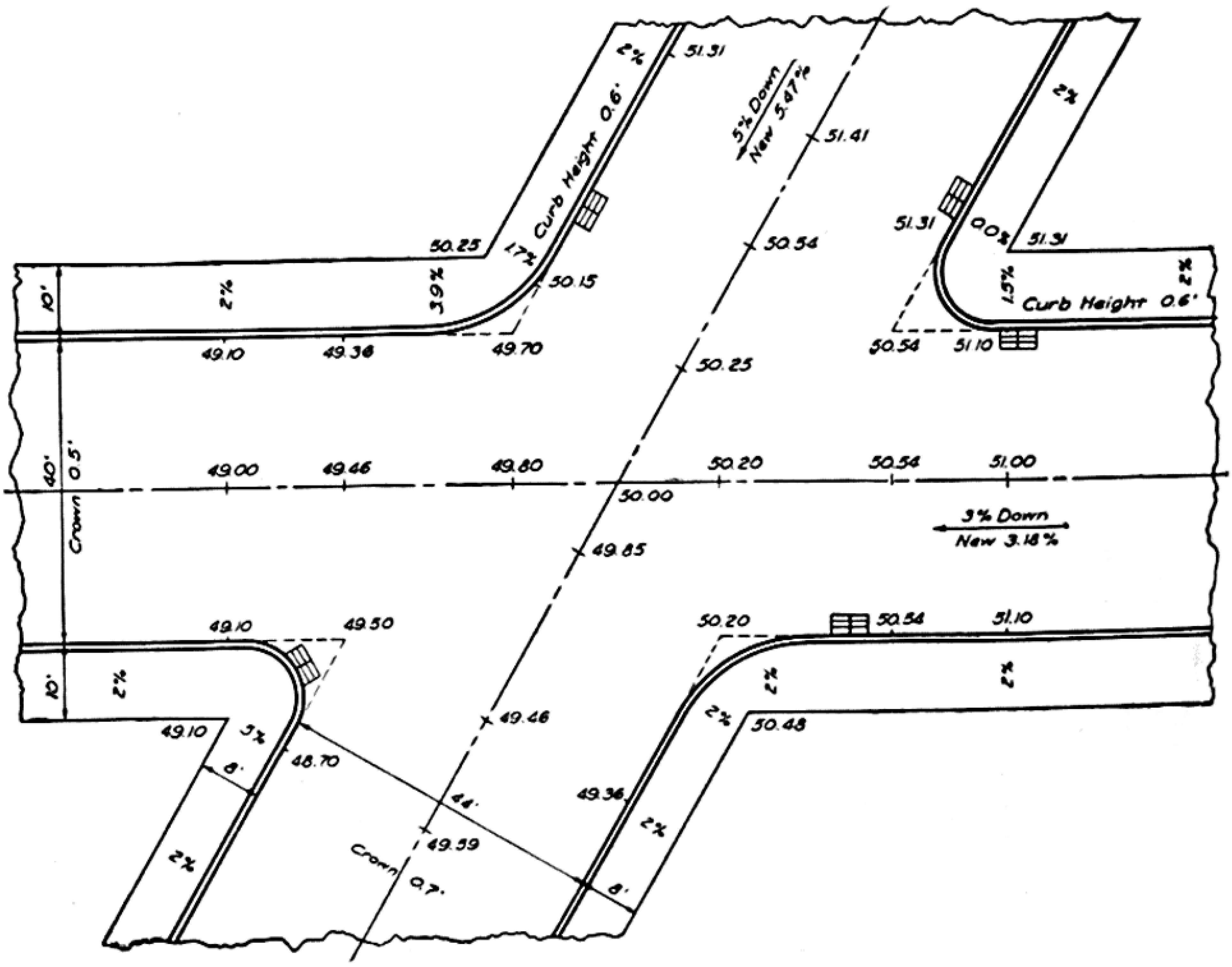


FIG. 65.—Design of a skew intersection.

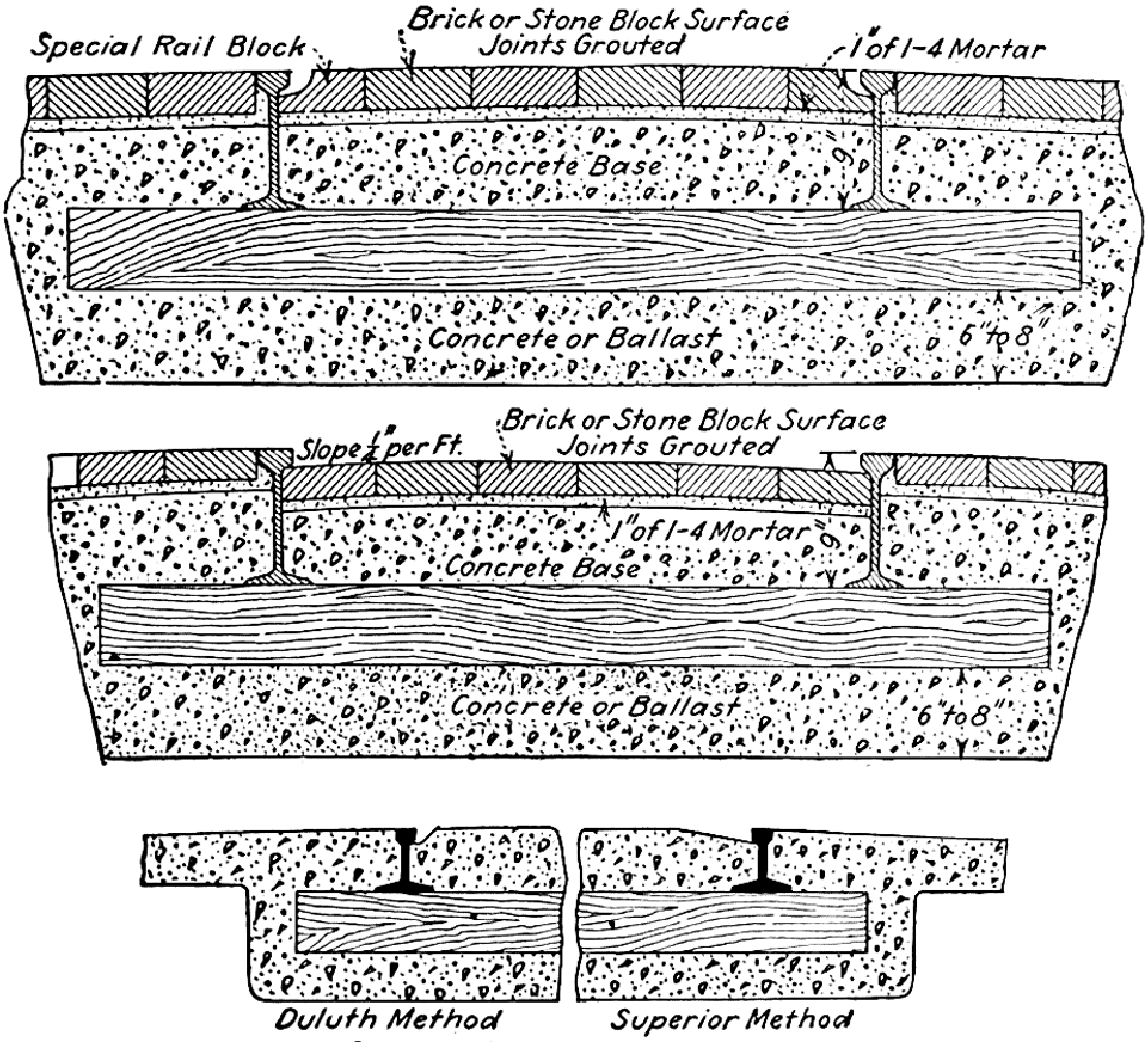
very durable car-track pavement. Thus we often find that on streets with sheet pavements (sheet asphalt, asphaltic concrete, bitulithic, etc.) the car-track area is paved with either stone blocks or vitrified brick.

Even for moderate-traffic streets paved with sheet surfaces, it is common to use a "toothing" along each rail, even though a sheet surface is laid between the rails. This toothing consists of one or two rows of stone blocks or vitrified brick laid along the rail with the long dimension parallel to the center line of the track.

Types of Car-track Rails.—The type of rail appears to be an important factor in the life of the car-track paving. Three types, or modifications of them, are employed. If the rail is not ade-

quately supported, the continual vertical motion as the cars pass will gradually loosen the paving blocks adjacent to the rail. Once loosened, water enters and softens the supporting soil under the ballast, hastening the deterioration of both track and pavement.

The effect of any movement of the track is increased if the paving blocks extend under the head of the rail.



Concrete Paving for Car Tracks

FIG. 66.—Paving along T-rails.

T-rails.—The T-rail is of the form that is used for steam roads, except that the rail is higher so as to permit the pavement being placed above the ties and to afford a more rigid support for cars. Where the rail is of this type a special shaped paving block is often employed to form a groove for the car-wheel flanges. This is not a satisfactory type of car-track paving for streets of heavy vehicular traffic because drivers will permit the wheels of vehicles to travel in the groove next the rail, and the concentration of traffic soon wears out the paving. Figure 66 shows the arrange-

ment of the paving along rails of this kind. The T-rail is more economical for the street-car company as a rule.

Grooved Rails.—The shape of this rail is best shown by Fig. 67 which also shows the arrangement of the paving along tracks of this kind. Since the top of the rail is even with the pavement surface, there is no tendency to guide the wheels of vehicles along the track.

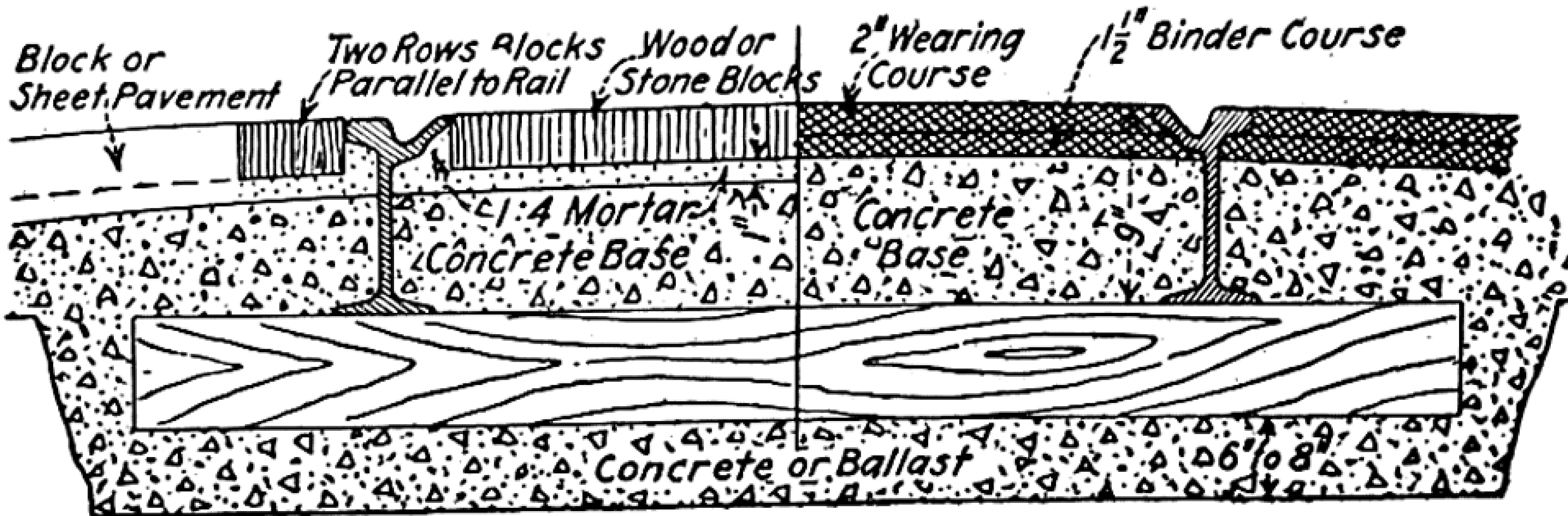


FIG. 67.—Paving along grooved rails.

Lipped Rails.—This rail is similar to the grooved rail, as will be seen by reference to Fig. 68.

Track Construction.—Regardless of the type of rail used, increasing attention is being paid to the track construction and especially to the foundation. The thickness of the ballast under the ties varies with soil conditions but is rarely less than 6 in. and often is more than 1 ft. The difficulties encountered with car-track paving are generally due to insufficient stability of the track. Figures 66, 67, and 68 show the practice in car-track construction on paved streets and represent the minimum requirements permissible if a stable track structure is to be obtained.

EXAMPLES OF GOOD PRACTICE[1]

1. Definition of Platforms.—The center-line intersection shall be deemed to be the point of intersection of the center lines except for cases where the center lines do not meet at a common point when it shall be the area included within the center lines at their intersection.

The curb-line platform shall be deemed to comprise the area included within the lines connecting the points on intersection of the curb tangents, or in the case of a street terminating at another street it shall comprise the area within the prolongations of the curb lines across the intersection and a line joining the curb tangents.

The building-line platform for rectangular intersections shall be deemed to include the area bounded by the prolongations of the building

[1] MOON, VERNON S., *Proc.* Municipal Engineers of the City of New York, 1911.

lines of both streets across the intersection so as to comprise the greatest platform area. In the case of other than right-angled intersections, it shall comprise the area bounded by the respective lines of each street and by lines at right angles or normal to the center lines and passing through acute-angled building-line corners, or the corners giving the greatest platform area. If the intersection of the center lines falls without the building-line platform, as above described, the said platform shall be increased sufficiently to include the said intersection. When the building-line corner is turned with a curve the platforms above defined shall be indicated upon the map unless herein definitely fixed.

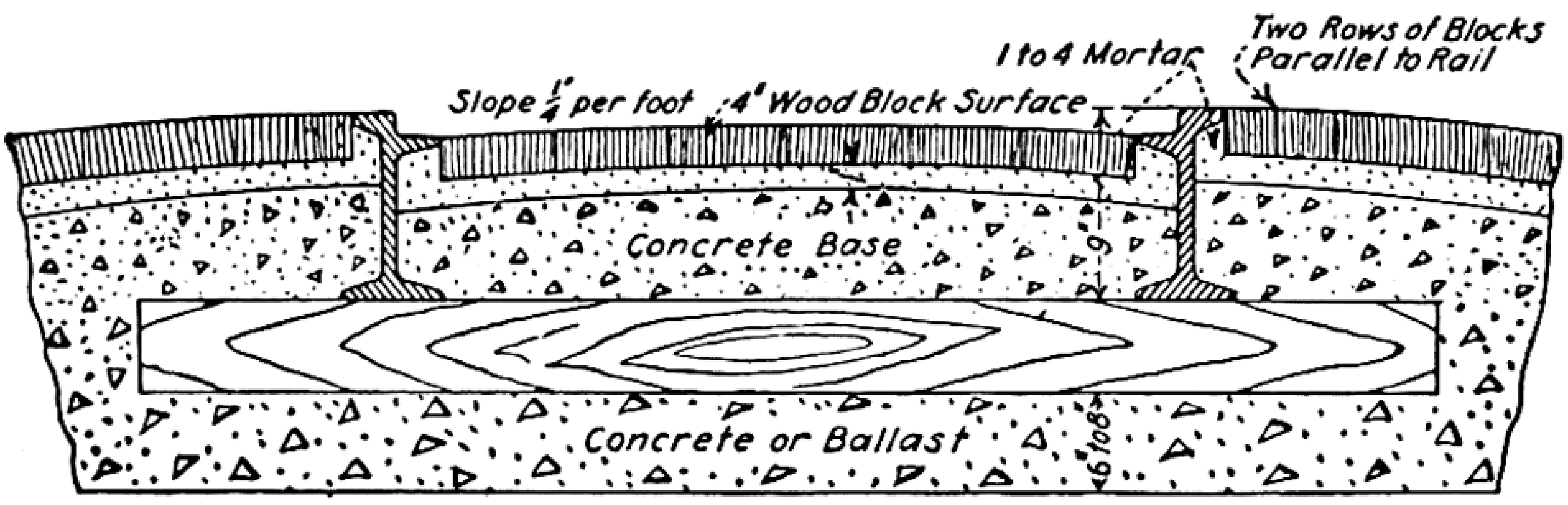


FIG. 68.—Paving along lipped or side-bearing rails.

2. Definitions of Elevations Fixing Grades.—Unless otherwise indicated on the map, the elevation shown at a street intersection shall be deemed to be that fixed for the point of intersection of the center lines of both streets affected, or for the center-line intersection.

3. Treatment of Center-line Intersection.—The center-line intersection, when it comprises an appreciable area and unless otherwise shown on the map, shall have a uniform elevation at its boundaries; and in determining the elevations for the other platforms herein described, the center-line intersection referred to as a basis of calculation shall be deemed to be the nearest point on the center line of each street at the boundary of the said platform.

4. Treatment of Platform for Streets Having a Light Grade.—If the grade of each of the interesecting streets is 3 per cent or less, as determined by calculating the rate between the established elevations, the elevation of the center lines of each street within the limits of the curb-line platform shall be the same as that fixed for the center-line intersection. The elevation of the curbs shall be determined as indicated in Paragraph 8. Provided, however, that the difference in the elevation of points on the center lines opposite any building-line corner shall not provide a greater transverse sidewalk slope than that fixed as the maximum in Paragraph 7, in which latter event the building-line platform shall be used and the grades of that portion of the streets adjoining the said corners shall be flattened between the boundaries of the building-

line platform and the center-line intersection, as provided in Paragraph 5(*a*).

5. Treatment of Platform for Streets Having a Steep Grade or Meeting at an Acute-angled Intersection.—(*a*) If the grade of any portion or portions of intersecting streets adjoining a building-line corner is over 3 per cent, as calculated between the established elevations, or if a further flattening of the platform grade is required to provide proper sidewalk slopes, for any part of an intersection described in Paragraph 4, the grades of the said portion or portions of each street shall be reduced between the boundaries of the building-line platform and the center-line intersection as follows: If the intersecting streets are of the same width, the grade of the street traversing the shorter block length adjoining the intersection shall be reduced one-third, and that of the street traversing the longer block shall be reduced two-thirds. In case the streets have different widths, the grade of the wider street shall be reduced one-third and that of the narrower street two-thirds between the above limits. All grades less than 3 per cent which are not herein required to be flattened shall be applied at the same rate as originally computed between established elevations. Provided, that in no case shall the maximum platform and sidewalk slopes fixed in Paragraphs 6 and 7 be exceeded.

Any excess in grade over that allowed in Paragraph 7 shall be removed by further flattening, as follows:

(*b*) Special flattening of platform grades for extreme cases of steep grades or acute-angled intersections. If the difference in elevation tentatively fixed for points on the center lines of intersecting streets opposite any building-line corner, after applying the minimum and up to the maximum transverse sidewalk slope on the higher and lower sides respectively, exceeds the maximum transverse sidewalk grades hereinbefore described, the elevation of each street at the boundary of the building-line platform shall be adjusted to remove the excess, the adjustment of each of the said elevations being directly proportional to the grade of each as originally flattened or applied.

For all cases covered by Paragraphs (*a*) and (*b*) the elevations at the intersections of the center line of each of the narrower streets or at the streets traversing the longer blocks, if they are of equal width, with the curb-line platform of the intersected street shall be the same as the elevation of a point on the center line of the intersected street immediately opposite the first-named intersection, except that the elevation at this point shall be abandoned when the grade along the center line between the curb-line platform and the building-line platform exceeds the grade as originally computed.

The grades of the center line of the wider street or of the street traversing the shorter block, if they are of equal width, shall be uniform

between the exterior boundaries of the building-line platform and the center-line intersection, except that the maximum platform slope hereinafter fixed shall not be exceeded. The grades of the center line of the narrower street or of the street traversing the longer block, if they are of equal width, shall be uniform between the elevations fixed at the exterior boundaries of the curb-line platform, and also between the latter point and the center-line intersection.

6. Maximum Platform Grades.—The maximum allowable grade along the center line between the curb-line platform and the center-line intersection shall be at the rate of 4 per cent, unless otherwise indicated on the map.

The grades along the center line between elevations established within the limits of a building-line platform shall be uniform, subject only to the flattening provided for in Paragraph 5 (*b*).

7. Transverse Sidewalk Grades.—Whenever practicable, the sidewalk shall slope upward in a direction at right angles to the curb toward the building line at the rate of 2 per cent.

The elevation of the sidewalk at the building-line corner shall be determined by applying this rate to the elevation of the curb giving the higher building-line elevation, at a point immediately opposite the corner, unless the resulting grade on the lower side exceeds 6 per cent, in which case the sidewalk shall be level on the higher side and a greater transverse sidewalk slope up to the maximum shall be used on the lower side.

The maximum transverse sidewalk slope shall be 6 per cent except in those cases where the street grade as originally computed on any street adjoining a building corner is more than 6 per cent, when the maximum slope shall be 10 per cent for either street, opposite the said corner. In no case shall the sidewalk at the building line be lower than that of a point immediately opposite it on the curb.

If the transverse sidewalk slope at the building-line corner is more or less than 2 per cent, it shall be made to agree with this latter rate at a point distant 25 ft. from the building-line corner.

8. Curb Elevations.—The relation between the elevation of the center lines and of the top of the curbs at points immediately opposite it at the boundary of and outwardly from the building-line platform shall be as follows: For roadway widths of 24 ft. or less the top of the curbs shall be 0.1 ft. higher than the center line. For roadway widths ranging from 24 ft. up to and including 34 ft. the top of the curbs and the center line shall be at the same elevation. For roadway widths ranging from 34 ft. up to and including 44 ft. the top of the curbs shall be 0.1 ft. lower than the center line. For roadway widths ranging from 44 ft. up to and including 54 ft. the top of the curbs shall be 0.2 ft. lower than the center line, and for roadway widths ranging from 54 ft.

up to and including 64 ft. the top of the curbs shall be 0.3 ft. lower than the center line.

The elevation of the intersection of the curb tangents shall be determined from a point immediately opposite on the center line of the wider street or the street traversing the shorter block, if they are of equal width, subject, however, to the same correction in elevation between the top of the curbs and the center line as herein provided.

9. Depth of Gutters.—Whenever practicable a standard depth of gutter of 0.4 ft. shall be used.

10. Curb Grades at Corners.—The tangents in the curbs shall be graded uniformly between the elevations established for them at the boundaries of the building-line platform and at the intersection of the curb tangents. The curve formed in the curb joining the tangents shall follow a uniform grade between the elevations of the curb tangents at the points of curve.

11. Grades between Platforms.—The grades of the center line and of the curbs between the elevations computed at platform intersections, or between a platform and an intermediate established elevation, shall be uniform.

CHAPTER VIII

ORDINARY AND TREATED EARTH ROADS

The portion of a highway system that comprises roads of purely local importance usually serves only a small volume of traffic, although the mileage of such roads in a state may be large. These roads comprise the land-access roads for areas not served by the main roads and consequently afford the outlet for products of the farm, the timber, and many of the small deposits of minerals.

The Ordinary Earth Road.—Although the land-access roads are important, the small volume of traffic and the relatively meager financial resources of the official body responsible for their maintenance often preclude the expenditure of more than $2,000 to $4,000 per mile for draining, grading, and surfacing. As a consequence a vast mileage of these is ordinary earth roads built of the soil naturally existing on the right-of-way.

The ordinary earth road can be maintained in usable condition in humid areas only during the portion of the year when precipitation is not too great and the destructive effects of freezing cycles do not have to be combated. At best this type of road provides a fine surface to travel over; at its worst it is an impassable mire. In between it may be dusty, slippery, or frozen into a rough though usable condition.

For 25 years or more, road builders have been following the will-o'-the-wisp of an inexpensive treatment of soil to convert the ordinary earth road into a surface of all-weather serviceability. The quest for a treatment that will accomplish the desired results has been unceasing, but the best that can be said as to the results so far attained is that progress has been made.

The first responsibility of the road builder is to construct the best road possible from the natural soil of the location and if it is inadequate for the service demanded to apply some treatment that will increase the serviceability, if funds are available for a treatment of proved merit. Where the funds are to be had, experimental sections may be constructed as a preliminary to the general adoption of a method.

Nature of the Earth Road.—Since soils are of so many kinds (Chap. IV), the characteristics of untreated earth roads may be expected to vary greatly from place to place and, from the very nature of the material, to be quite sensitive to the effects of climatic conditions. The problem of the road builder who must provide earth roads for certain locations is to secure the most serviceable road that is possible under the conditions and with the type of soil available.

The engineer's problem is to forecast, insofar as he is able, the effects of climate upon the soil of the site that he plans to improve and to construct the surface in a manner that will minimize the effects of climate. Fortunately, the climatic influences are generally uniform, and the soil conditions are easily determined in the area encompassed by the ordinary highway improvement project; but plans that are satisfactory in one locality are not necessarily suitable in some other faraway district. The important thing is to know what methods, if any, are available to the road builder to minimize each kind of climatic influence in the construction and maintenance of the earth road intended for light traffic.

The earth road is sometimes graded and drained, as the first step in the construction of a surfaced highway; in that case the design will be based on the principles set forth in Chap. VI; and although that phase of earth-road building will be introduced occasionally, this chapter is intended to deal with an altogether different type of construction.

DESIGN

The general principles of the design of rural highways have already been presented at length (Chap. VI), but the application of these to the light-traffic land-access and local service roads involves an understanding of the traffic conditions on these roads and the financial limitations imposed in planning their improvement.

Basis of Design.—The land-access (secondary) road is usually designed for traffic densities not exceeding about 200 vehicles per day, with the average daily traffic probably less than 100 vehicles per day. Many sections of these roads will have as low as 50 vehicles per day, and some sections, except on rare occasions, will carry only a dozen or so per day. Obviously this classifica-

tion excludes the main market roads, sometimes called "county trunk highways," and state trunk-line roads. The problems of design, then, are imposed by the necessity of securing serviceable roads for a small volume of traffic at a cost that in many instances must be kept down to less than $3,000 per mile. It is recognized that an ordinary earth road cannot be expected to afford all-weather service in humid areas, but by careful drainage and under good maintenance it can be made serviceable for all but the most adverse months of the year.

Cross-sections.—A typical cross-section for an earth road is shown in Fig. 69. The width of the roadway will seldom be less than 24 and may be as great as 36 ft., depending upon the volume and character of the traffic. Agricultural machinery of one sort or another and loads of produce on racks will be moved over these

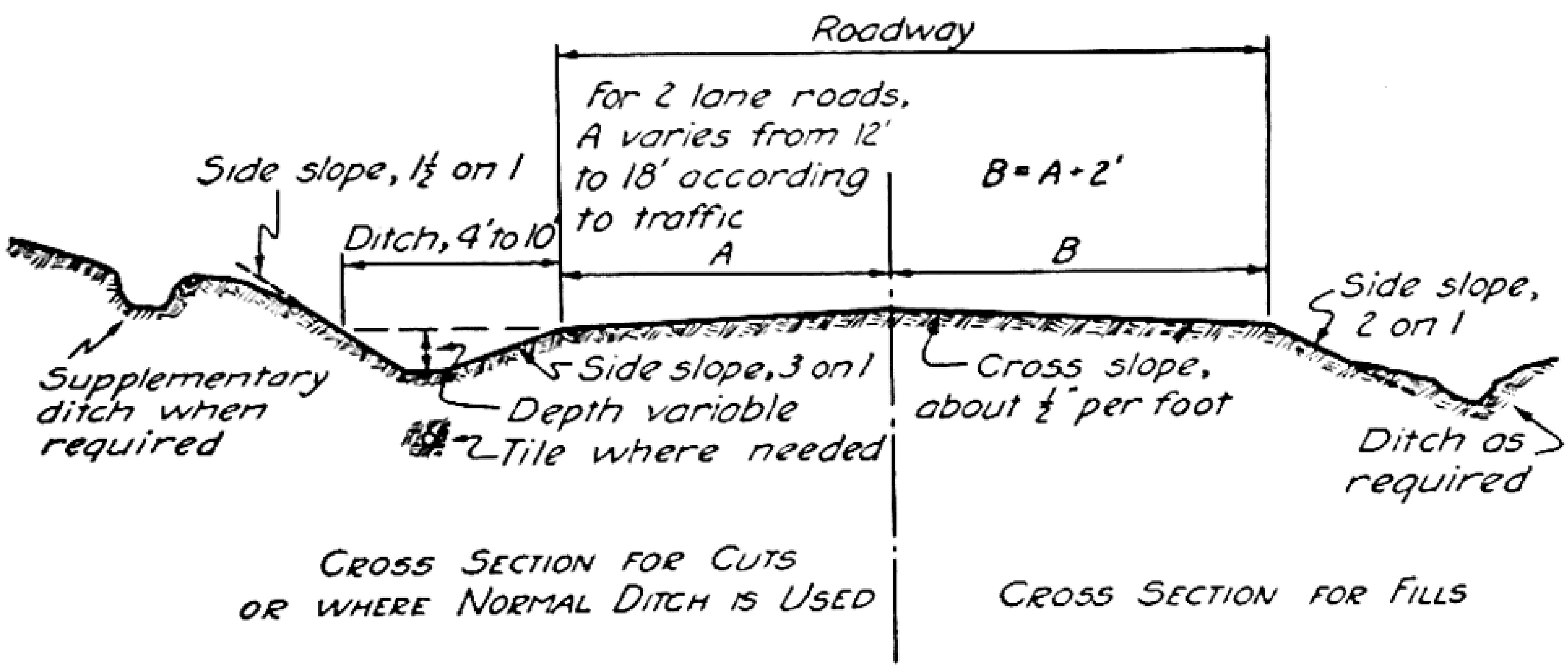


Fig. 69.—Typical cross-section for an earth road.

roads, and a reasonable width is required even though the full width is used only infrequently. Ample ditches are provided for surface drainage, and back slopes in cuts are treated as cheaply as is consistent with reasonable stability. The cross-slope on the traveled portion is kept down to about ½ in. per foot or even less. Fills are built with the maximum side slope that the soil will stand, which is generally about 2 on 1 near the top and a little less on the lower part of the slope.

Grades.—The longitudinal grades on highways of this class are fixed at the *maximum* permissible for the traffic, and in rolling country the grade line will follow closely the undulations of the land. The most elementary computations of the economics of grades will show that very little gross saving to traffic will result from the construction of *minimum* grades on these

highways, and expenditures for providing grades below those required for travel in high gear by automobiles usually cannot be justified economically. Beyond this general principle the grade reduction planned will be determined by safety considerations and drainage requirements. For example, a little grading will sometimes eliminate the need for a culvert; or material for raising low-lying, poorly drained stretches may most conveniently be secured by reducing an adjacent hill. But care should be taken not to remove the weathered top soil, which is generally the most stable part, from a hilltop to be used elsewhere only to find that a layer of unstable soil had been uncovered. A study of the soil profile (page 98) will disclose whether or not grading to a proposed grade line is likely to uncover underlying unstable soil.

Drainage.—In humid regions, rain and snow produce a volume of water that must be disposed of expertly, or it will be very detrimental to the serviceability of the plain earth road. The freezing-and-thawing cycle inevitably lowers the serviceability of water-saturated soils. Drainage in the broad sense will minimize these effects. The first requisite is good surface drainage, by which is meant a system of roadside ditches that will really keep the surface water on the move but provide ample temporary storage capacity for water at times of heavy rains. It is surprising the frequency with which sections of road are encountered where water is impounded in roadside ditches because of the inadequate longitudinal slope of the ditch. Without exception this is due to failure actually to construct the ditch to a surveyed gradient. Ditches graded by machinery without guide grade stakes are seldom wholly effective.

The drainage problem presented by melting snow is a different story. At the season of the year when snow begins to melt, the ditches are likely to be blocked with snow and ice, and for a period of several days the snow water cannot readily follow the established surface drainage channels and will flow on the road surface or wherever it can find a channel. In some cases supplementary tile will aid in expediting the removal of the snow water. The best protection of the road lies in building the traveled way high enough above the surrounding land so that the snow will blow off the surface pretty generally. In cuts, there will always be some accumulations of snow even though snow fences are installed, but it is hardly feasible to use snow fences generally

on the feeder roads, which are the ones under discussion here. In prairie country in the colder areas the snow blows about continuously through the winter months, and cuts will be filled as rapidly as they are cleared. A high grade-line is very helpful in such areas, as the road surface will to a considerable degree be windswept clear of snow.

Sand Drifts.—The method employed for minimizing snow difficulties is effective for regions where sand and fine soil particles drift much as does snow. A high grade-line, that is, 2 or 3 ft. above the general level of the land, is desirable, and cuts should be avoided at all cost. If they must be used, it must be recognized that they will have to be cleared frequently.

Swamp Roads.—Swampy areas are encountered in connection with road building in many parts of the world, and if the road is to be surfaced with natural soil brought in from borrow pits, there are two plans of construction from which to choose the one considered the better for the project in hand.

The older and more widely used method is to float the road crust on the underlying unstable soil by means of a mat of logs or brush, the old familiar corduroy method. This method is also often employed for roads to be surfaced with gravel. On top of this mat is placed a layer of soil thick enough to provide a smooth surface. Unless the cover is a mixture of soil and gravel, it will probably soften sufficiently at times to permit wheels to cut through to the mat. At such times the road is, of course, well-nigh impassable to motor traffic and is anything but satisfactory for animal-drawn traffic; but this cannot be avoided with the earth-covered corduroy.

Another method of constructing roads through swamps is to fill across the swamp, settling the fill by means of explosives so that the embankment rests on the stable soil that lies below the water and the plant growth that constitutes the swamp (page 84). Sometimes the depth is so great that the cost of filling is prohibitive, but in many cases the swampy deposit is relatively shallow, and this method is quite feasible. The method of settling is simple, although good judgment is needed in locating and charging the holes. The general plan is to dump fill material into the swamp, bringing the road embankment approximately to grade. Holes are then cut into the fill with an earth auger, and charges of dynamite placed at the bottom of the layer of freshly placed fill. When these are exploded, the blast forces the soft material

from under the new fill and allows it to settle down on to stable ground. A few trials will show how the charges should be placed and the best explosive to use. Commercial 40 per cent dynamite is used quite successfully. This method is applicable to the construction of heavy-duty roads as well as to the feeder roads.

Grade Reduction.—When the design provides for lowering the grade on a section of road, certain preliminary work should first be completed. If there are weeds, brush, or trees on the portion of the road to be graded, these should be cut away or grubbed out, and the resulting debris burned. It is especially undesirable to leave under a fill quantities of organic matter, because it will decay and cause unequal settlement and may lead to undercutting from water that works under the fill where it is porous from the decay of organic matter.

If fills are made on side hills where the cross-slope is steeper than 4 on 1, provision should be made to insure a bond between the new fill and the old ground. If the existing soil is porous and the slope does not exceed 4 on 1, this can be accomplished by plowing a series of furrows about 2 ft. apart, parallel to the center line of the fill. For dense soils or slopes greater than 3 on 1 the area to be covered by the fill should be graded to a series of horizontal benches about 4 ft. wide. After this preparation is completed, the fill may be placed in a series of horizontal layers not over 2 ft. thick.

When the fill is constructed on a road that has a cross-slope flatter than 4 on 1, the new material may be placed without special preparation of the site, except to remove the vegetable matter. For fills of this class it is undoubtedly advantageous to place the fill material in layers not exceeding 2 ft. in thickness. The teams and scrapers will compact the material as they travel over it, and the fill that is built up will not settle excessively. Some engineers require that each layer shall be rolled before the succeeding layer is placed, but it is doubtful if there is enough benefit from the rolling to compensate for the cost on roads that are not to be surfaced, which is the class under discussion.

Prevailing practice in earth-road construction is to permit the fill to be placed without restrictions as to the thickness of the layers and without rolling. Suitable provision must be made for probable shrinkage of embankment in any case.[1]

[1] This discussion is restricted to ordinary earth roads. The method is entirely different if a wearing course is to be added (page 98).

Blade Grader Work.—The blade grader is used mainly for work involving the moving of earth from the edges of the road to the middle to form the typical road berm. This machine is not efficient for moving earth lengthwise of the road, except for short distances, and then only when small quantities are used to fill occasional low places.

The blade grader favored for construction work is a heavy machine with a blade 12 ft. long. The power is a tractor of the size known as the "10-ton," by which is really meant any one of a number of diesel-engine powered tractors weighing from 7½ to 10 tons. The "track-laying" types have quite generally superseded the wheel-type tractors. Often, two blade graders hitched in tandem are drawn by a single tractor.

In recent years self-contained blade grader units embodying the power plant and the grader mechanism have come into wide use. They may be diesel- or gasoline-engine powered and are always on rubber-tired wheels, often of the "six-wheel" type. They are more widely used for maintenance and finishing operations than for road shaping.

Elevating Grader Work.—The elevating grader[1] is a combined excavator and loader which may be expected to load as much as 150 cu. yd. of material per hour, when working conditions are favorable, although the average operator probably does not load much over 100 cu. yd. per hour. It is an economical machine to use in grade reduction work if the cuts are long enough to enable the machine to operate without too much loss of time in turning. The grader is operated in a loop, cutting and loading along the parallel sides of the loop and idling while making the turns at the ends of the cut. The economical use of the elevating grader is impossible on loops less than 300 ft. long.

The older types of elevating grader take power from the rear wheels for the operation of the elevator belt. This is never very economical but is practicable and dependable where the soil affords good traction. In wet or light soils there is a good deal of slippage at the driving wheels of this type of grader which reduces the output of the machine quite materially. Modern elevating graders are constructed with a power drive for the elevating belt, provided either from the tractor by means of a power take-off shaft or from an engine mounted on the elevating

[1] THEE, T. C., "Performance of Key Equipment Used in Highway Construction," *Roads and Streets*, Vol. 78, No. 9, p. 291, October, 1935.

grader. This system of driving practically doubles the plow capacity of the grader.

The excavated material falls from the elevator belt into dump wagons or trucks for transportation to embankment. When a

FIG. 70.—The elevating grader on grade reduction work. (*Courtesy of U.S. Public Roads.*)

vehicle is loaded, the grader is halted until an empty vehicle is in place, then moves forward until another vehicle is loaded, and the cycle of operation is thus repeated. It will be apparent that good management is necessary to insure the efficient operation of an outfit of this sort. The more salient points to take into consideration are the following:

The elevating grader should not be adopted for jobs where the cuts are so short that a high percentage of time will be required in turning at the end of the loop.

The grader should be provided with ample power. If animal-drawn, at least 20 good horses or mules should be used. Although animal power is now almost obsolete in the United States, it is still used in many places outside the United States. If tractor-drawn, the track-laying type should be used, and its speed should be adjusted to the grader which ought to be hauled at a rate between $2\frac{1}{2}$ and $2\frac{3}{4}$ miles per hour.

The job should be carefully analyzed, and the correct number of trucks or wagons secured, so that the grader does not lose time because of inadequate transportation. The correct number of trucks can be estimated on the basis of the length of haul and the average speed of travel. For short hauls, dump trucks of 5 to $7\frac{1}{2}$ tons capacity with track-laying tread and drawn by a tractor are widely used. For the longer hauls the 1- or 2-yd. dump trucks of standard type with pneumatic tires are generally used.

Scraper Work.—When grading involves removing material from short cuts and shallow cuts, the volume of earth to be removed from any one cut may be insufficient to warrant the use of the elevating grader, and some form of scraper will be employed. Three types of scrapers are available for this purpose.

The wheel scraper,[1] which has a capacity of about 10 cu. ft., is used when the excavated material is to be moved from 300 up to 1,000 ft. It is drawn by two animals, and an extra team is used for loading. The soil must be plowed before it can be loaded, and the common practice is to plow two furrows, then remove the loosened material, and repeat. Efficiency of use depends upon securing a full load every time and keeping the scrapers moving in a loop of the minimum length that will deliver the material and especially adjusting the loading and dumping places so that the length of the loop does not vary too much.

The fresno[2] scraper is used when the haul is less than 500 ft. and especially for loops of not over 200 ft. The scraper has a capacity of about 8 cu. ft. and is drawn by three animals. It is

[1] HARRISON, J. L., "The Economical Use of Wheel Scrapers," *Public Roads*, Vol. 5, No. 10, p. 16, December, 1924.

[2] HARRISON, J. L., "The Cost of Grading with Fresnoes," *Public Roads*. Vol. 5, No. 8, p. 10, October, 1924.

used for work similar to that for which the wheel scraper is employed but on shorter loops.

Here, again, the trend is away from animal-drawn equipment, and the "bulldozer," a tractor "pusher" blade grader with a trailing scoop grader, is most widely used for a short-haul movement of excavated material.

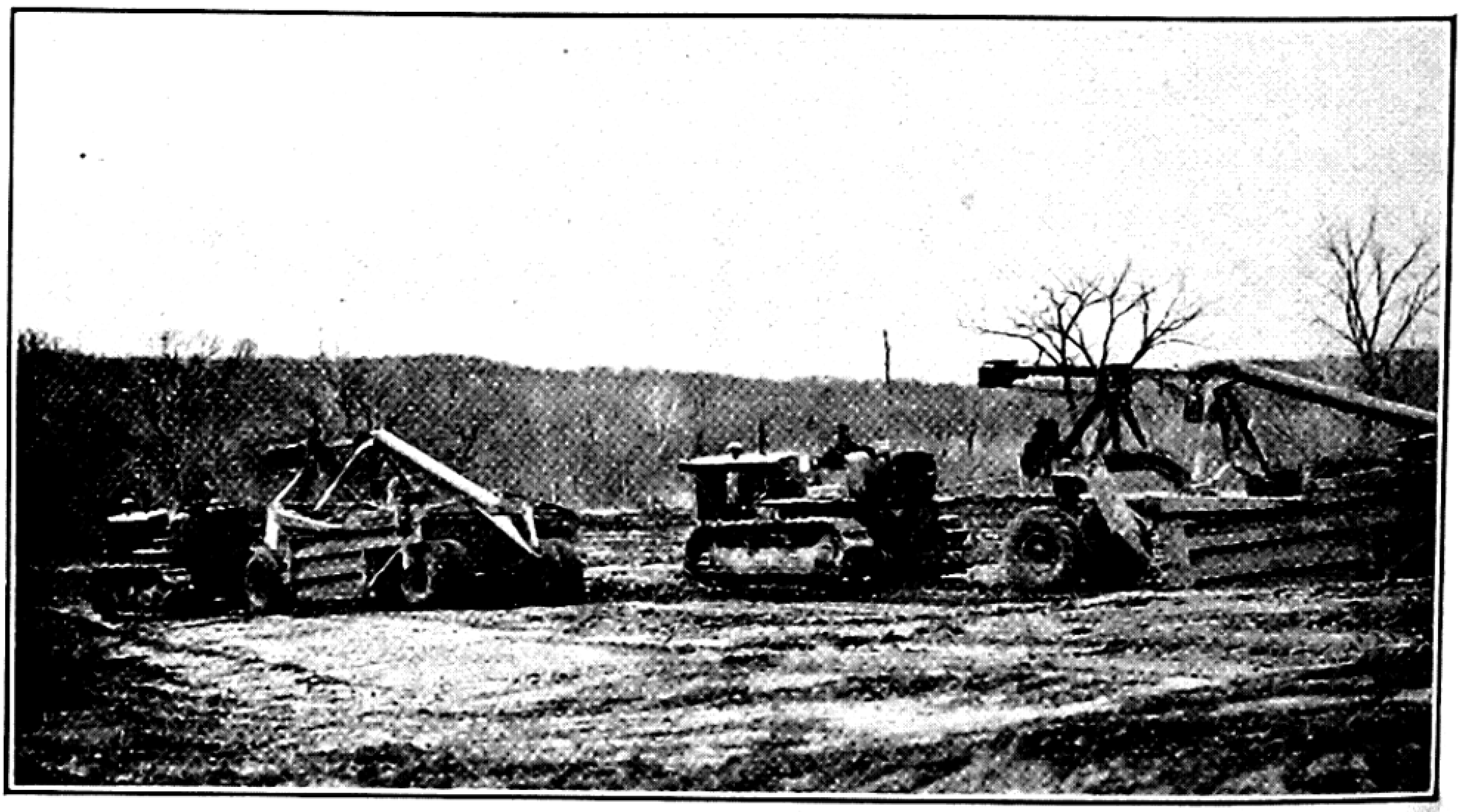


FIG. 71.—The drag scraper of large capacity is used for grade reduction work.

Steam-shovel Excavation.—Where heavy grading[1] is being done so that an outfit can be used continuously for a long time without frequent costly moves, the steam shovel is economical. A standard railroad outfit would be employed for very heavy work, and the materials handled in dump cars on the industrial railway. For lighter work the traction type of shovel would be more satisfactory, and the earth hauled in tractor-drawn dump wagons or power dump trucks of some one of the modern designs.

These grading methods are also adaptable to the earthwork in connection with the construction of the heavy-duty roads.

Maintenance.—It is to be expected that a newly constructed earth road will be unstable because of lack of solidity in embankments and lack of uniformity of texture in the soil exposed in the cuts. Weathering, settlement, and precipitation are continuously at work, and their effects must be counteracted by suitable maintenance. The work of maintenance must begin as soon as

[1] ALLEN, T. WARREN, "Power Shovel Operation in Highway Grading," *Public Roads*, Vol. 8, No. 12, p. 251, February, 1928.

the road is placed under traffic and be continuous thereafter. The patrol system employing maintainer scrapers of some kind, supplemented by emergency dragging, seems to produce the most satisfactory results. Under this system a section of road is taken care of by a man who devotes all his time to the duties incident thereto and who will use the equipment selected as being the best adapted to the soil and climatic conditions.

In regions of high rainfall the log or plank drag will be used to smooth the road surface while it is wet, thus filling the depressions and serving to squeegee some of the water off the traveled part of the surface. This tool does not compact the surface to any considerable extent, nor does it move enough mud and slush to fill the larger depressions. Moreover, the traffic will disturb the smooth surface readily and seemingly undo all the work performed by the drag. Continued dragging as the road surface is drying will insure that, when finally dry, the surface will be fairly smooth. This method is particularly useful for roads that carry very light traffic and where the soil is not too waxy and tough when wet. Some soils can scarcely be handled until they reach the crumbly stage of drying, and some others cannot be handled by the drag after they begin to dry out, because they are so waxy that the drag cannot move enough material really to smooth the surface.

Principle of Road Soil Treatment.—Although there are some soils that[1] do not require treatment to give them sufficient load-bearing capacity, the usual run of the soils encountered in road construction do require some treatment to provide a suitable road surface for year-round use. For a soil to be stable under loads it must possess a high degree of cohesion and considerable mechanical stability.

Cohesion is that property of a material which gives it high resistance to deformation because of the great mutual attraction between the very small (microscopic and even submicroscopic) particles from forces of the nature of molecular attraction. Mechanical stability is due to high internal friction in the mass due to the mutual support of the adjacent angular particles through static forces acting between particles too large to be affected by molecular forces. In addition, the soil must be low in permeability because the infiltration of free water supplies a

[1] HOUSEL, W. S., "Principles of Soil Stabilization," *Civil Eng.*, May, 1937, p. 341.

lubricant that facilitates movement of the particles and the moisture also creates a condition favorable to disruptive effects from freezing and thawing cycles. Capillarity is the near relative of permeability, and high capillarity brings in the water that may become ice lenses in the freezing and thawing cycle. Moreover, if a soil has high capillarity it is not susceptible of drainage by means of tile or other underdrains. The density of a soil is, subject to certain limitations, directly related to the load-carrying capacity. Density must not be carried so far that no volume change can take place without disruptive effects, since it is certain that there will be some volume change with variations in the moisture content.

Methods of Treatment.—The methods employed for treating soils to secure wearing surfaces that are superior to those of the natural soil roads fall into three groups.[1]

1. Mixing together two or more soils to provide a material having the desired particle-size distribution following the principles set forth in Chap. IV. The best mixture of the available soils is determined in the laboratory, and the soils are then brought to the road after it has been graded to the desired profile. One soil will, of course, always be the one of which the roadway is composed after the shaping has been completed. Other soils that are to be incorporated are dumped on the road and the upper 10 or 12 in. are thoroughly mixed and then compacted by a sheep's-foot or disk roller or by gangs of rubber-tired wheels mounted so that they can be weighted and tractor drawn. The mixing is performed by any one of several machines, each being satisfactory under certain conditions, and none doing good work under all conditions. The machines most widely used are the gang plow, certain types of multiple-blade cultivators, and heavy-disk harrows. Frequently the soil is sprinkled before rolling. Generally roads of this class are finished with a bituminous mat to provide a dustless, abrasion-resisting wearing surface. The load-supporting property is provided by the 8 to 10 in. of densified soil which retains its stability by virtue of the binding action of the moisture film in the mass and resists infiltration of excess (free) moisture because of low capillarity

[1] These processes are often referred to as methods of soil stabilization; but the word "stabilization" has been applied to so many schemes of doubtful value that its use is avoided herein.

and low void space. Good surface and subsurface drainage must be provided for surfaces of this type when built in humid regions.

This method of construction is more widely applicable to subgrades for wearing surfaces of appreciable thickness such as bituminous gravel and macadams 3 to 6 in. thick than it is to soil that is intended to be used as a wearing surface. It is applicable to the construction of wearing surfaces for light-duty land-access roads in many regions, where financial resources preclude the construction of watertight wearing surfaces.

2. The incorporation of a bituminous binder into the road surface. The asphalt in the binder serves to glue together the soil particles and to fill the void space with a water-resistant material. Sands and very sandy loams can be treated successfully by this method. The other types of soil consist of particles to which the asphalt does not readily adhere, and the operation of mixing the bituminous binder and soil is very troublesome; no wholly satisfactory method has as yet been devised.[1] The methods of dry mixing, wet mixing, and impregnating have been attempted with varying success. In some areas fairly durable roads have been developed by this process, whereas in others it seemed impossible to achieve complete waterproofing.

3. The addition of portland cement to the soil, thus providing bond by means of crystals forming in the spaces between the particles of soil to cement them together. Trial mixtures of the soil and portland cement are consolidated and then subjected to alternate wetting and drying, and freezing and thawing. The amount of cement required for a suitable mixture is thus established, and the data made available for estimating the cost of the work.

The soil-cement road is a mixed-in-place type. The soil is loosened for the depth required to give the desired compacted thickness; the cement, added and mixed with the soil by means of plows, disk harrows, or multiblade cultivators. At the proper stage of mixing the road is sprinkled, and the final mixing is on the wet material. Compaction is by means of the sheep's-foot or disk roller.

The resulting surface has considerable resistance to climatic effects and high load-carrying capacity but if subjected to direct

[1] "Soil Stabilization with Asphalt," Construction Ser. 40, The Asphalt Institute, New York, N.Y., Aug. 1, 1938.

wear is likely to ravel in places and gradually become uneven. For long service it should be provided with a bituminous mat to serve as a wearing surface.[1]

4. The introduction into the soil mass of a deliquescent salt such as common salt or calcium chloride, which supplements the capillarity of the soil in retaining the moisture which serves as the binding agent. The method requires that the soil be brought to a satisfactory particle-size distribution, as described in Method 1, and the addition of about 10 lb. of calcium chloride or salt per ton of soil, the exact proportion to be determined by laboratory studies of the materials to be employed.

The mixture may be prepared in a stationary mixing plant and hauled to the road or, as is the more common practice, mixed in place on the road.

The surface mixture thus prepared is suitable as the base for a wearing surface but will not be a satisfactory wearing surface. The most common process is to employ a mixed-in-place bituminous mat on the surface.[2]

5. The use of chemical reagents to bring about a surface condition on the soil particles that is favorable to the retention of soil moisture or binding agents in soils that otherwise prove to be unstable. This is a field that has just begun to be explored, and no very conclusive investigations have as yet been reported.[3]

Although rapid progress is being made in the development of methods of increasing the load-carrying capacity of soils by the various treatments described above, none is very effective in increasing the ability of the soil to resist wear. The cheapest and most effective method so far devised for insuring a reasonably satisfactory wearing surface for the light-traffic treated earth

[1] MILLS, J. H., "Road Base Stabilization with Portland Cement," *Eng. News-Record*, Nov. 28, 1935, p. 751.

LANCASTER, C. M., "Base Stabilization with Portland Cement," *Roads and Streets*, December, 1938, p. 17.

"Soil-cement Mixtures for Roads," *Proc. 17th Ann. Meeting*, Highway Research Board, Part II, December, 1937.

[2] STEWART, L. C., and S. J. WHITE, "Premixed Stabilized Soil for Road Surfaces," *Eng. News-Record*, p. 389, Sept. 19, 1935.

"Salt Stabilized Road Practice Developing Rapidly," *Eng. News-Record*, July 4, 1935, p. 11.

[3] WINTERKORN, HANS, "Adsorption Phenomena in Relation to Soil Stabilization," *Proc. 15th Ann. Meeting*, Highway Research Board. 1935, p. 343.

roads is to incorporate gravel or crushed stone in the surface. This is done by placing a window of the gravel along the edge of the roadway to be dragged on to the traveled way a little at a time as a part of the maintenance operation. It is dragged on only when the road surface has been softened by rain and the gravel will be incorporated in the surface by the traffic. If the traffic is so heavy that such a surface is disagreeably dusty, a bituminous mat is applied by the mixed-in-place process.

The cost per mile for treated earth roads two lanes in width varies greatly and may be as low as $5,000 per mile in regions where conditions are favorable to twice that amount in other regions.

CHAPTER IX

SAND-CLAY, TOPSOIL, AND GRAVEL ROADS

Sand-clay and topsoil road surface are identical in essential properties and in physical appearance, each consisting of a wearing surface of soil mortar of a particular grading. Gravel roads are a distinct type that, however, depends for stability upon the soil mortar employed as a binder. It seems logical to group these types for the purpose of discussion.

Definitions.—Certain terms employed in the literature dealing with these types are frequently used inexactly and therefore are defined herein according to what appears to be the most common usage of the term.

Sand-clay Road Surface.—A sand-clay road surface consists of a layer of soil mortar in which the ingredients in predetermined quantities are mixed in place on the road.

Topsoil Road Surface.—A topsoil road surface consists of a layer of soil mortar of suitable composition obtained from the surface of the fields in the vicinity of the road.

Gravel Road Surface.—The gravel road surface consists of a layer of natural gravel, to which soil mortar may or may not be added to augment the binding action of the soil mortar naturally contained in the gravel.

Soil Mortar.—Soil Mortar is a term employed to designate that portion of a soil or mixture of soil and gravel that will pass the No. 10 standard sieve. The limits of the several sizes of grains in a soil mortar that is suitable as a binding agent or a wearing surface are shown in Fig. 72.

The Nature of Soil Mortar.—Soil mortar is a natural or artificial mixture of sand, clay, silt, and possibly some colloidal material. The mechanical analysis of soil mortars in the types of sand-clay road surfaces found in the southeastern part of the United States[1] is indicated in Table XIX.

[1] STRAHAN, C. M., "A Study of Gravel, Top-soil and Sand-clay Roads in Georgia," *Public Roads*, Vol. 10, No. 7, p. 118, September, 1929.

TABLE XIX.—COMPOSITION OF SAND-CLAY AND TOPSOIL MIXTURES

Fraction	Hard, or Class *A*	Medium, or Class *B*	Soft, or Class *C*
	Per Cent	Per Cent	Per Cent
Clay	10–18	15–25	10–25
Silt	5–15	10–20	10–20
Total sand	65–80	60–70	55–80
Total	100	100	100
Sand retained on No. 60 sieve	45–60	30–45	15–30

The economic field for the sand-clay and topsoil road surfacing[1] is in improving roads where the traffic averages between 400 and 600 vehicles per day and where the cost of the initial con-

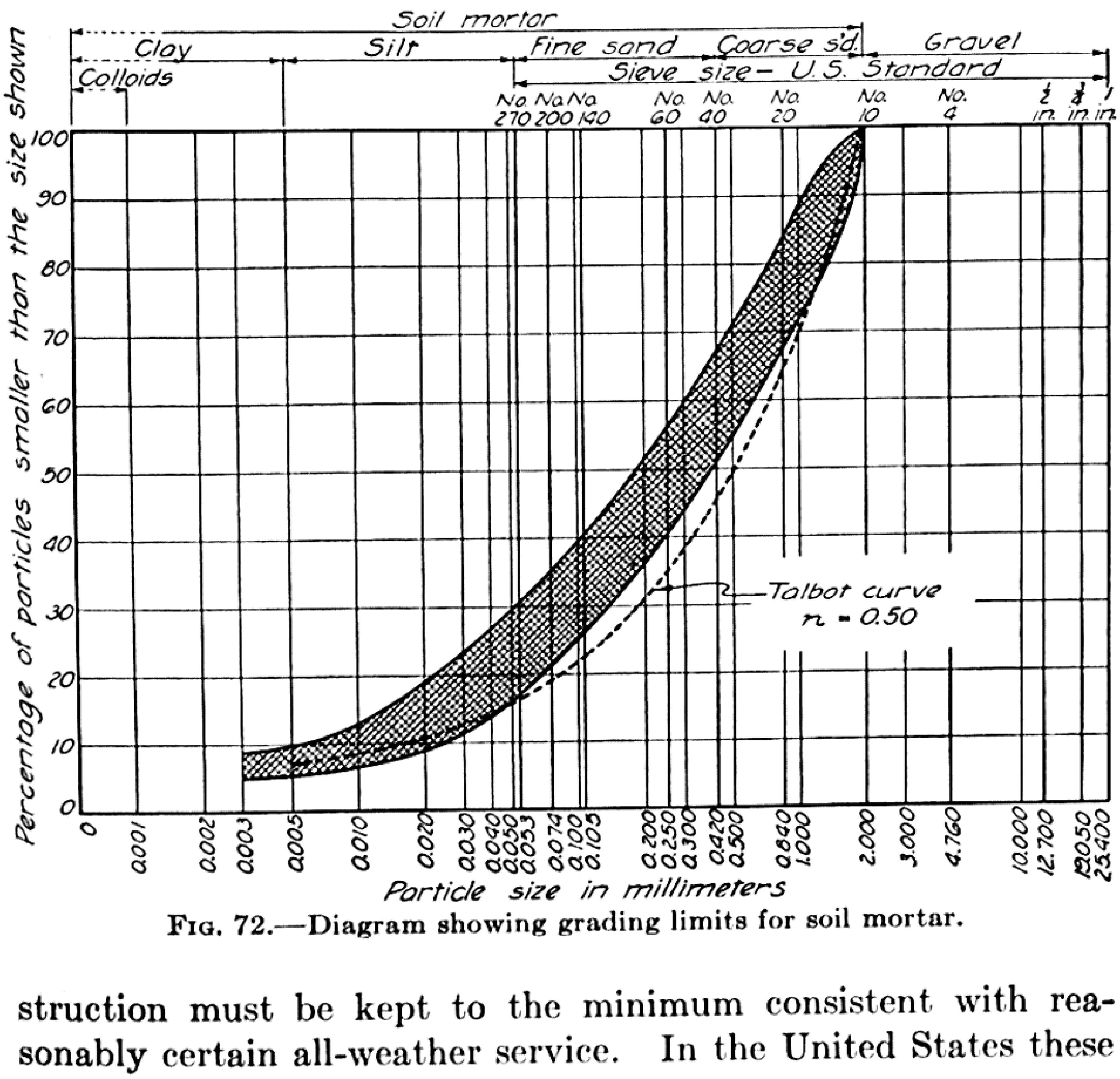


FIG. 72.—Diagram showing grading limits for soil mortar.

struction must be kept to the minimum consistent with reasonably certain all-weather service. In the United States these

[1] CONNOR, C. N., "Low Cost Improved Roads," *Proc.* 17th *Annual Meeting*, Highway Research Board, Part II, 1928.

types have been used most widely in the South Atlantic states. It is not always possible to find or mix a soil mortar of Class *A* (Table XIX). With that as an ideal, however, the engineer will proportion the available material to secure the best mixture possible. The more nearly he succeeds in approximating the grading of the Class *A* mixture the more likely he is to have a stable road surface.

Nature of Gravel.—The natural gravels occur in many parts of the world[1] and consist of deposits of the familiar mixture of partly rounded and smoothed fragments of rock, ranging in size from fine grains of sand to sizable boulders. Usually there is some earthy material mixed with the gravel, and sometimes the earthy material comprises a considerable percentage of the deposit. The portion of the material that will pass the standard No. 10 sieve is termed "soil mortar," and the portion coarser than sand is called "gravel" in the nomenclature of soils mechanics. In ordinary parlance the term gravel is applied indiscriminately to the natural mixtures of soil mortar and gravel and to any fraction screened out of such materials.

Gravel deposits differ widely in the relative proportions of sand and pebbles, the amount and grading of the soil mortar, and the character of the rock of which the fragments are composed. Deposits of fissured limestone, nouvaculite, or granite are sometimes called gravel, but in these the pieces are angular instead of rounded, and such material is not usually thought of as true gravel.

The gravel road is usually adopted for a particular project because gravel of acceptable quality can be obtained at low cost or in the hope that local supplies can be utilized. The ideal road gravel consists of fragments of durable rock with a grading approaching that of the Talbot curve shown in Figs. 21 and 72. It will be noted that the clay comprises from 5 to 7½ per cent of the whole and the soil mortar (passing a No. 10 sieve) from 30 to 50 per cent. Durable roads can be constructed from a wide variety of natural gravels if the proper amount of fine material is provided, but the more exacting the traffic the greater the need to secure gravel approaching closely the ideal in properties. The range of particle-size distribution within which good results may be expected is shown in Figs. 72 and 75.

[1] PAULS, J. T., "Materials for Low Cost Roads," *Eng. News-Record*, Dec. 3, 1931, p. 879.

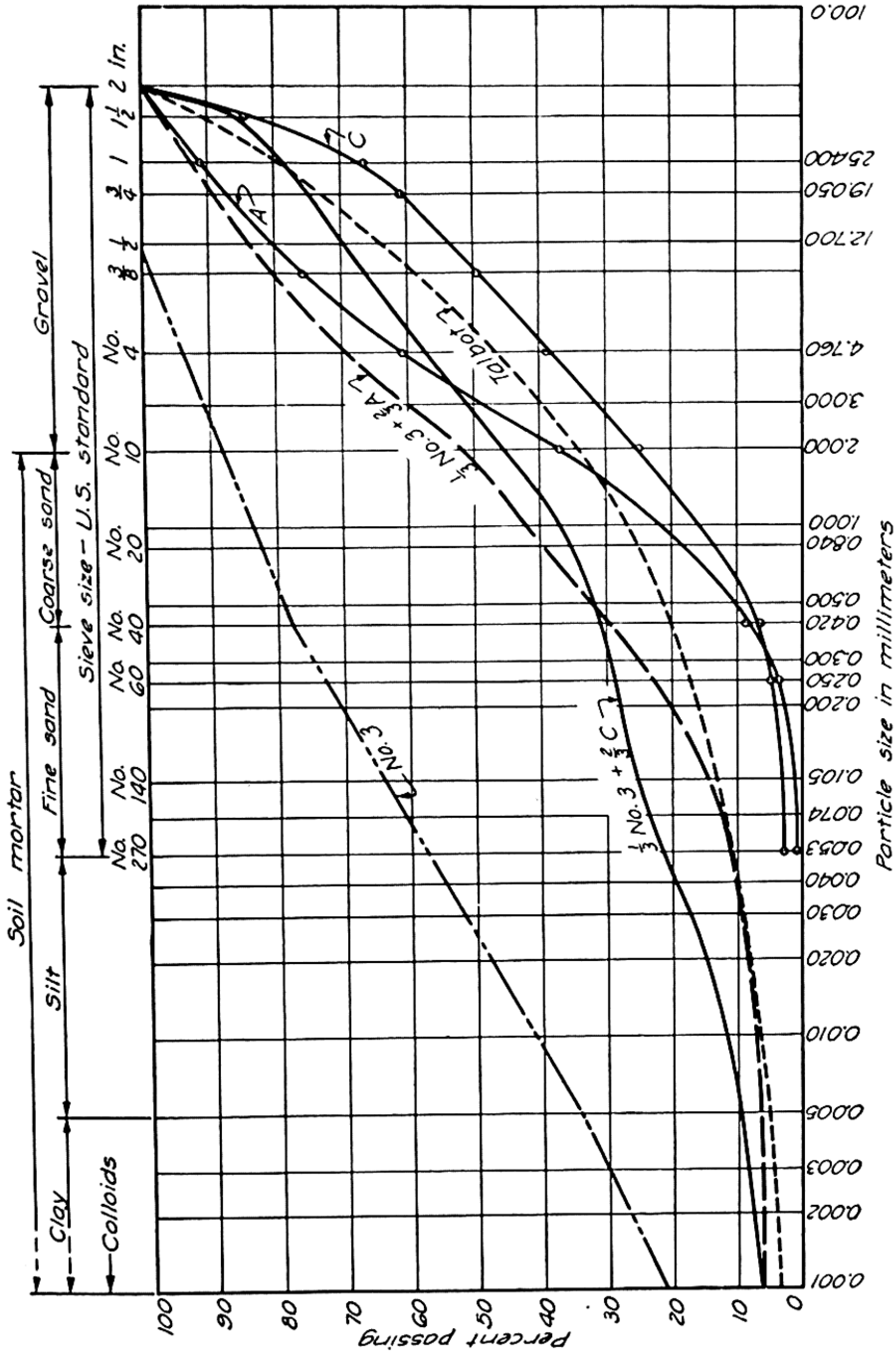


FIG. 73.—Typical particle-size distribution curves for soils.

The discussion of proportions introduces a confusing and contradictory system of nomenclature because this same gravel may be used for fabricating concrete, and the several fractions into which it is separated by laboratory analyses are: passing $1\frac{1}{2}$-in. screen and retained on the No. 4 sieve, called "coarse aggregate," "stone," or "pebbles"; passing the No. 4 sieve, called "sand"; portion removed by the standard elutriation test (about 200-mesh and finer), called "silt" and "clay." In the nomenclature of the gravel road the portion coarser than the 10-mesh sieve is called "gravel," and that finer than 10-mesh is called "soil mortar." The portion of the soil mortar finer than 0.005 mm. is called "clay."

SAND-CLAY AND TOPSOIL ROADS

Construction on Sandy Roads.—When the natural soil of a road is of a sandy character, and particularly where it contains very little loam or clayey material, a method of construction has been developed that consists in adding selected soil to the sand in such proportions as are needed to produce a good soil mortar. The method of construction is relatively simple, since with a deep sandy soil there is no problem of instability due to underground water. The grading involves taking care of the surface drainage including the necessary weirs or other devices in the ditches for preventing ditch erosion. The subgrade of sand is graded to a cross-section without cross-slope, and upon it is placed a sufficient amount of the selected soil to form a layer of soil mortar about 12 in. thick before compaction. The selected soil is dumped on the road in the correct proportion for the layer that is to be mixed, and then the sand and clay are mixed by means of ordinary earthworking tools. The method that seems to have been used quite extensively is first to turn over the layer with an ordinary grading plow. This is accomplished by plowing through the clay down into the sand layer and turning up sand with the clay. This operation is repeated several times followed by mixing with disk harrows and finally by dragging the mixture repeatedly back and forth across the roadway with a blade grader. Although the mixing operation appears to be a simple one, except for the amount of labor involved, it is a most neglected factor in the construction of this type of roadway surface. The success of the type depends upon selecting the correct materials and proportions and then securing thorough mixing.

In many areas the only material available for this type of construction is a natural soil mortar, found on the surface of fields, which already contains all the sand that can be held together by the finer grained material in the mixture. In such cases the sand-clay surface is constructed on top of the sandy road by the addition of a layer of this topsoil which is placed without incorporating it with the underlying sand. These surfaces are called "topsoil roads" and will be described more fully in a later section.

Construction on Clay Roads.—Clay roads may be improved by the addition of the proper amount of granular material to the surface as described on page 230. If more permanent and serviceable surfaces are desired and can be financed, the clay road may be converted into a sand-clay or topsoil road. The conversion to a sand-clay surface is accomplished by adding to the clay surface a sufficient quantity of selected sand to produce a layer of soil mortar of acceptable grading having a thickness of about 12 in. when compacted. Generally the clay soil is loosened by plowing; the sand is added in the proper amount and mixed with the clay in much the same manner as the sand-clay surfaces built on sandy soils. The plow, disk harrow, and blade grader are utilized in the mixing process.

In many cases, as with sandy roads, the only material available for stabilizing the clay surface is surface soil from near-by fields which is really a natural soil mortar of acceptable grading and naturally contains about all the clay binder that is desirable. Under such circumstances a topsoil wearing surface is placed on the clay road. The subgrade upon which the topsoil mixture is placed is graded with a slight crown, and the topsoil is placed and spread to the ditch line, usually by the feather-edge method whereby the middle thickness before compaction is about 12 in. and gradually diminishes to 4 or 5 in. at the shoulders. The topsoil mixture is dumped on the road, the lumps thoroughly broken up, and then the layer is remixed to insure uniformity. It is then spread with the blade grader and allowed to compact under traffic.

Characteristics of the Sand-clay and Topsoil Surfaces.—It must be apparent from the description of the methods of construction that the sand-clay and topsoil road surfaces will not reach maximum compaction and serviceability until subjected to a considerable amount of traffic. It has been the usual experi-

ence that such roads require a full season of traffic before they can be said to be of the normal texture and stability. In the meantime, there will be periods when these surfaces will be quite unstable, and repeated shaping with the blade grader or other maintenance equipment is necessary to keep them smooth and in serviceable condition. However, the kneading action of traffic constitutes a mixing process which goes on continuously and will eventually compact this type of surface sufficiently to insure reasonable serviceability. A well-built sand-clay road looks very much like an ordinary clay road, but upon close inspection it will be found that the surface consists of a large percentage of particles of fine sand. After prolonged rains it may soften to a depth of ½ in. or more but does not appear to be disturbed very much under the wheel loads that are ordinarily to be expected on roads of this type. It is a type that gets very dusty in prolonged dry weather, and the loss of material from the surface in the form of dust during dry weather amounts to perhaps as much as ¾ in. or more per season. The cost of these roads ranges up to $3,000 per mile and perhaps under the most favorable condition amounts to at least $1,500 per mile.

Topsoil Surfaces.—The topsoil surface is a sand-clay surface constructed of a natural mixture of materials that comprises the weathered surface layer of the fields in extensive areas of the South Atlantic states and to a lesser extent in small areas throughout the humid regions of the world. These topsoils are of the composition prescribed for soil mortar suitable for road surface construction (Fig. 72 and Table XIX). Generally a layer about a foot thick is stripped from the field and used without further treatment except for the mixing that takes place in placing and shaping the road surface. This type of road is therefore a true sand-clay surface, and the name "topsoil" merely indicates the source of the material.

Surface Oiling of Sand-clay Roads.—The sand-clay road becomes very dusty in dry weather, and this is not only disagreeable to the users but also results in serious loss of the fine material from the surface of the road. Experiments have been proceeding for a long time with a view to developing a bituminous surface treatment suitable for these roads, and a reasonable amount of success has finally been achieved. The methods employed will be described in detail in Chap. XV.

Sand-clay and Topsoil Roads in Georgia.—The life history[1] of road-soil surfaces reflects the combined results of original surface composition, depth, cross-section, and width used, methods of mixing and consolidation, and continuous upkeep by road machines and is typical of the experience with these types when they have been constructed properly and well maintained. If the road soils selected lie within certain recognized limits of composition and are subjected to a traffic of moderate intensity (400 to 600 vehicles per day), the average quality and length of service are highly satisfactory, the maintenance operations are quickly and easily performed, and the cost of both construction and maintenance is gratifyingly low.

These types have a large and permanent place in highway betterment in three directions:

1. As the first stage of surfacing for newly graded main roads.
2. As the chief dependence for surfacing secondary roads.
3. As a valuable subgrade under higher types of surface.

The following comments on the composition and classification of road soils are suggestive of the correct practice:

1. The mechanical analysis as at present used seems a satisfactory laboratory method in the premises, and the definitions of coarse material, sand, silt, and clay are in the main acceptable.

2. Knowledge of the differences in adhesive values of clays is insufficient. The way has been opened to supply this deficiency in accord with tests suggested by Dr. Charles Terzaghi for the identification of clays.[2]

3. Until a more accurate knowledge of the clay ingredient is available, the present limits of composition for Classes *A*, *B*, and *C* soil mortars (Table XIX) may be allowed to stand.

4. In judging these materials, full emphasis should be placed upon the soil mortar, *i.e.*, material below the No. 10 sieve. Weak soil mortars even with large amounts of "coarse material" often do not give proper stability under traffic. In general, Class *C* mortars (Table XIX) are not to be recommended except for very light traffic or when the surface is to be covered with a 2-in. layer of gravel, semigravel, or coarse screenings.

5. "Coarse material" above 10 per cent in amount distinctly increases the stability and durability of the surface less with Class *C* and progressively more with Class *B* and Class *A* mortars (Table XIX).

6. Coarse material is most effective when present in graded sizes from 1 in. downward. Such material of micaceous, feldspathic, or slaty types is most commonly soft and soon becomes valueless.

[1] STRAHAN, *op. cit.*, pp. 134–136.

[2] TERZAGHI, CHARLES, "Principles of Soil Mechanics," *Eng. News-Record*, Vol. 95, *et seq*, Nov. 5, 1925. (Eight Articles.)

7. Organic matter gives much initial adhesive strength but is soon oxidized or blown away.

8. Of the total sand in a road soil mortar, the portion that lies above the No. 60 sieve is a most important factor in hardness, supporting value, and durability of the surface, especially under wet conditions. A soil mortar of this type will not give satisfactory service unless it contains a liberal percentage of sand coarser than the No. 60 sieve, except in the case of cherts.

This type of construction is acceptable within certain rather well-understood limitations which may be summarized as follows:

1. This road surface is best adapted to light or moderate traffic densities which these research data place at 400 to 600 vehicles per day according to composition of the surface and to the provision for constant patrol maintenance. Adequate equipment for an intelligent execution of maintenance work has much to do with both the quality of service rendered and the efficient life of such roads.

2. The expectancy is an effective life of 6 to 8 years under 400 to 600 vehicles daily, with a quality of service of 75 per cent, an annual distributed cost per mile of $500, and an operating index of $1 per daily vehicle per mile per year.

The economic aspects of the use of these types of surfaces may be summarized as follows:

1. Road soils such as semigravel, topsoil, natural sand-clay, artificial sand-clay mixtures, iron-silica pebble deposits, and cherts are available over large areas of the state. Their normal cost at present contract prices may be estimated within limits of $1,800 to $2,500 per mile for a 26-ft. surface, averaging 7½ in. compacted depth and requiring 3,400 cu. yd. of loose material per mile.

2. The normal life expectancy of such surfaces with a traffic of 400 to 600 vehicles per day may be taken at 6 to 8 years before replacement.

3. The normal provision for maintenance should be at least $200 per mile per year.

4. The annual interest charges at 5 per cent will be $90 to $125 per mile for the initial cost limits suggested to be amortized as depreciation occurs.

5. A satisfactory annual distributed cost per mile range is from $450 to $650 per mile per year with a corresponding operating index close to $1 per year when a traffic density of 400 to 600 vehicles per day is reached. This gives a highly satisfactory service quality expressed as 75 to 80 per cent.

6. Traffic above 800 vehicles per day increases the maintenance cost and lowers the life expectancy to a marked extent except with semigravel surfaces containing more than 25 per cent of coarse material.

7. The data secured by the 5-year study of road soil surfaces is deemed sufficient to establish the economic basis suggested for these types.

GRAVEL ROADS

The gravel road is a wearing surface of natural gravel, with or without an admixture of other materials to aid in retaining the

moisture necessary to bind the aggregate into a stable mass. This is one of the oldest and most widely used of road surfaces and is familar to everyone who travels the highways. Many so-called gravel roads are composed of granular materials other than gravel, such as burnt shale from mine refuse dumps, low-grade ores, shell, slag, and even crushed stone. This section will be confined to a discussion of the true gravel road.[1]

Principles of Construction.—The gravel surface is intended to be a wearing surface that will be stable notwithstanding seasonal variations of climatic conditions and therefore must consist of granular material whose particles can withstand the wear of traffic and are not affected adversely by water. The granular material must be cemented together to provide density and stability, the binding agent being the surface tension of water held in the mass when the proper grading is secured by adding soil to the gravel, if it does not in the natural state contain enough soil mortar to provide the requisite grading. Since soil mortar is effective as a binder only when it contains the correct amount, "optimum," of moisture, a deliquescent salt may be incorporated to aid in the retention of moisture in the gravel layer. The available gravel may naturally contain enough soil mortar to provide the bonding agent, but generally it does not, and soil of a suitable composition must be added.

1. When the surface is constructed on the "traffic-bound" theory, the additional clay is provided by the subgrade soil, which is incorporated in the gravel mass by the mixing action of traffic and the maintenance operations. This is a considerably hit-or-miss process, and the results may be quite disappointing.
2. Soil of suitable composition is added to the gravel layer after it has been placed on the road and mixed with it by repeatedly windrowing and spreading the gravel by means of the blade grader. On some projects, use has been made of a portable mixer which receives material at one end, mixes it, and delivers it at the other, moving along the subgrade in a continuous operation.
3. The gravel and the required additional soil are machine mixed at a gravel pit or storage yard, then delivered to the road to be spread and compacted.

Wearing Qualities.—The hardness and toughness of the stones in the gravel may be determined by the standard abrasion test or by means of the Los Angeles rattler test (page 313). The

[1] MORRISON, ROGER L., "Secondary Road Surfaces," *Eng. News-Record*, Jan. 16, 1936, p. 78.

results are neither so uniform nor so significant as for broken stone but serve as a reasonably accurate measure of the value of the material. Quite often, individual stones in the lot tested are of a partially disintegrated material and lose a disproportionate amount during the test. A gravel containing a few stones of this kind would probably wear better in the road than the test would indicate. Again, a selected sample would often show better in the test than the material from which it was taken would under traffic in a road. However, the wear of modern traffic is not troublesome. The distortion of the surface is the difficult problem to combat, but resistance to distortion is not dependent on the wearing properties of the gravel.

A careful examination of the gravel in a deposit will usually indicate, in a general way, whether or not it is durable enough for road purposes. If a large proportion of soft or partially disintegrated stones is encountered, the gravel should not be used unless the cost is low and the deposit contains a fair proportion of a good sand, when it can be used for what might be termed "coarse sand-clay construction."

Bonding Properties.—Gravel roads are said to be bonded with clay. Actually the bonding agent is water. The clay (*soil mortar* is the correct term to use), a part of which may be colloidal in form, provides a physical structure favorable to high surface tension on the part of the hygroscopic and capillary moisture. This moisture, by virtue of capillary tension, and perhaps other manifestations of surface tension and even molecular attraction, provides the bonding element in the gravel road. Thus, although it is not literally true that clay is the bonding agent, it is true that clay is utilized to create the condition whereby the bonding action takes place. It is not usually practicable to secure clay disassociated from other soil elements to mix with road gravels; on the contrary, the clay binder is obtained as an element in soil mortar. It will be noted that soil mortar consists of clay, silt, and sand, and there may be only a small percentage of clay in the available soil (Figs. 21 and 73). The addition of a suitable soil mortar to a gravel provides a mixture in which the particle-size distribution creates the condition of low porosity needed for the bonding action. As a rough check it is customary to consider that road gravels should contain 5 to 10 per cent of clay, and this is a good enough rule for the relatively thin traffic-bound surfaces constructed on the light-traffic local roads. For the

feeder roads carrying medium traffic and for gravel base courses to be surfaced with bituminous materials or mixtures, the gravel and soil mortar should be carefully adjusted to one of the particle-size distribution bands shown in Figs. 72 and 75. Mixtures that fall within the shaded bands are generally satisfactory, and those nearest the Talbot curve are best.

Bonding action due to crystal formation may be expected when certain ingredients are present in gravel. Calcium and magnesium carbonates will dissolve in water; and when the water evaporates, crystals will form. When gravel containing lime becomes saturated with water, some calcium carbonate will be dissolved in the water; and upon evaporation of the water, crystals of carbonate will be left attached to the surfaces in the minute interstices in the mass. The same phenomenon is manifested in the formation of stalactites. There are several minerals other than lime that may be involved in this type of bonding action, but, after all, this type of cementing action is of little significance in gravel road construction.

Selection of Gravel.—Since the desirable characteristics of the gravel to be used for road purposes are known, the one nearest the ideal may be chosen from among those available, or two gravels may be mixed on the work, and thus a material produced which will be superior to either alone. It often happens that gravel roads must be built from local material that is far from ideal, but such gravel can be made into a serviceable road by careful manipulation.

The pit gravels fall into three groups: those that approximate fairly closely the ideal gravel, those that are too coarse, those that are too fine. Any one of these may be deficient in bonding material.

If the gravel is too coarse, the larger pieces may be screened out and thrown aside or crushed and mixed with the finer material. It may be desirable in some instances to add soil mortar containing clay in appropriate proportions to increase the amount of bonding material and to facilitate compacting the gravel in the road. In other cases two gravels may be mixed to secure the grading desired. As an example of this process there are shown in Fig. 73 the particle-size distribution curves for a coarse gravel (No. 3 on the curve sheet) and two fine gravels (*A* and *C* on the curve sheet). Trial mixtures have been made up to produce a material having a particle-size distribution approaching

the Talbot curve. The results are platted for comparison with the Talbot curve in Fig. 73.

Gravel pits in some areas tend to run to an excess of fine material, and in such localities coarse material cannot readily be obtained to improve the mixture, but the excess fine material can be screened out, or the material may be employed for a sand-clay by suitable adjustment of the grading of the soil mortar fraction.

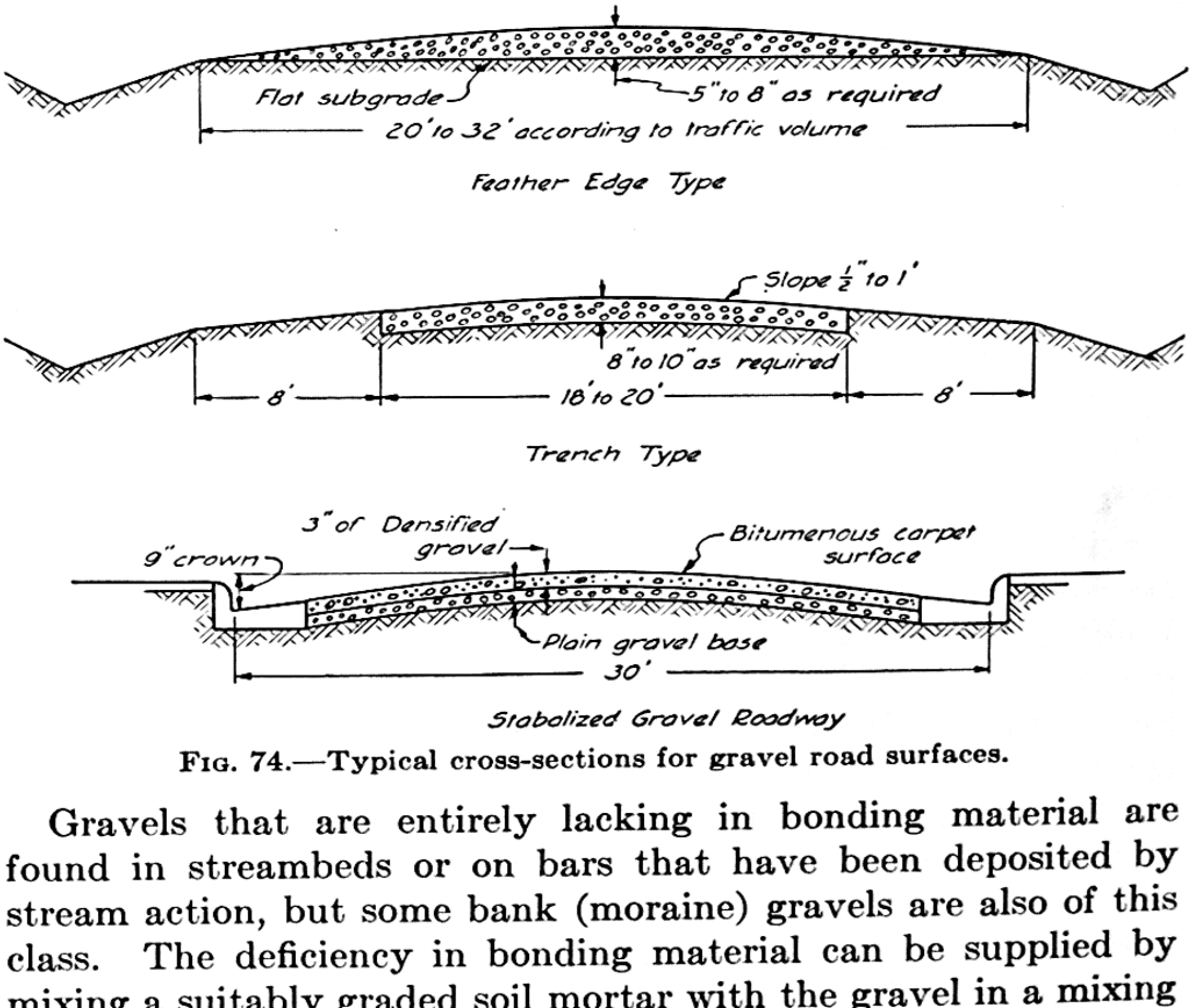


Fig. 74.—Typical cross-sections for gravel road surfaces.

Gravels that are entirely lacking in bonding material are found in streambeds or on bars that have been deposited by stream action, but some bank (moraine) gravels are also of this class. The deficiency in bonding material can be supplied by mixing a suitably graded soil mortar with the gravel in a mixing plant at the gravel pit or after the gravel has been spread on the road. If this is carefully done, excellent roads result, because gravels of this class are usually very durable.

Cross-sections.—The gravel surface is most frequently placed to a "feather-edge" cross-section as shown in Fig. 74. The thickness of the layer is varied according to the requirements of a particular project, which will depend upon subgrade soil, climate, and the traffic loads expected. The surface is sometimes placed by the trench method, also shown in Fig. 74, which provides for a uniform thickness for the full width of the gravel surface.

This method is especially adapted to plant-mixed materials, because the full layer may be placed and compacted in a series of consecutive operations as one project. If rolling is to be effective, substantial shoulder berms are required to prevent the mass from spreading unduly during the rolling. On street work, combined curb and gutter are provided at the edge of the roadway.

Preparation of Roadbed.—For local roads, by which is meant those carrying an average of not to exceed 100 vehicles per day, the preparation of the roadbed need not be elaborate. The necessary drainage ditches are constructed, and the grades are reduced to the desired extent. As a rule, little grade reduction is justifiable on this type of highway beyond that needed to reach the economical maximum plus grade. The profile usually follows the minor undulations of the land, and a width of about 24 ft. is graded to a level cross-section to receive the gravel.

For the more important gravel-surfaced highways the right-of-way is graded in accordance with the design developed in accordance with the principles outlined in Chap. VI. The portion to be covered with gravel is generally finished with a level cross-section and should be allowed to become fairly compact before the gravel is placed. A few weeks of travel accompanied by continuous maintenance will generally suffice, although 6 months of travel would undoubtedly produce a more stable foundation. In arid or semiarid regions there is nothing to be gained by delaying the placing of the gravel after the earthwork is completed.

If the trench method is to be followed, the earth is excavated to form a trench for the gravel, the excavated material being drawn to the sides of the road to form shoulders to retain the gravel. This method is sometimes followed where the carefully graded gravel mixture is to be rolled as it is placed and is recommended if the gravel road is expected to be anything more than a layer of loose gravel to be compacted slowly by the traffic.

The feather-edge method of placing the gravel requires no special preparation of the roadbed if the earthwork has been completed in the proper manner. The gravel is placed on the roadbed and feathers out at the edge, there being no berm to retain the layer. This type of surface cannot be rolled advantageously.

Thickness of the Gravel Surface.—The gravel road surface is generally classified as one of the flexible types of surfaces, and consequently the thickness of the layer of gravel is not computed

according to any mathematical rule. It is expected that the layer will gradually be built up through the addition of new material in the normal course of the maintenance operations. For the light-traffic type of surface the feather-edge section is used, and the thickness at the middle is about 5 or 6 in. after compaction. For the carefully proportioned mixtures intended for a considerable volume of traffic and placed on carefully prepared subgrades, the thickness is 8 to 10 in. These thicknesses can be

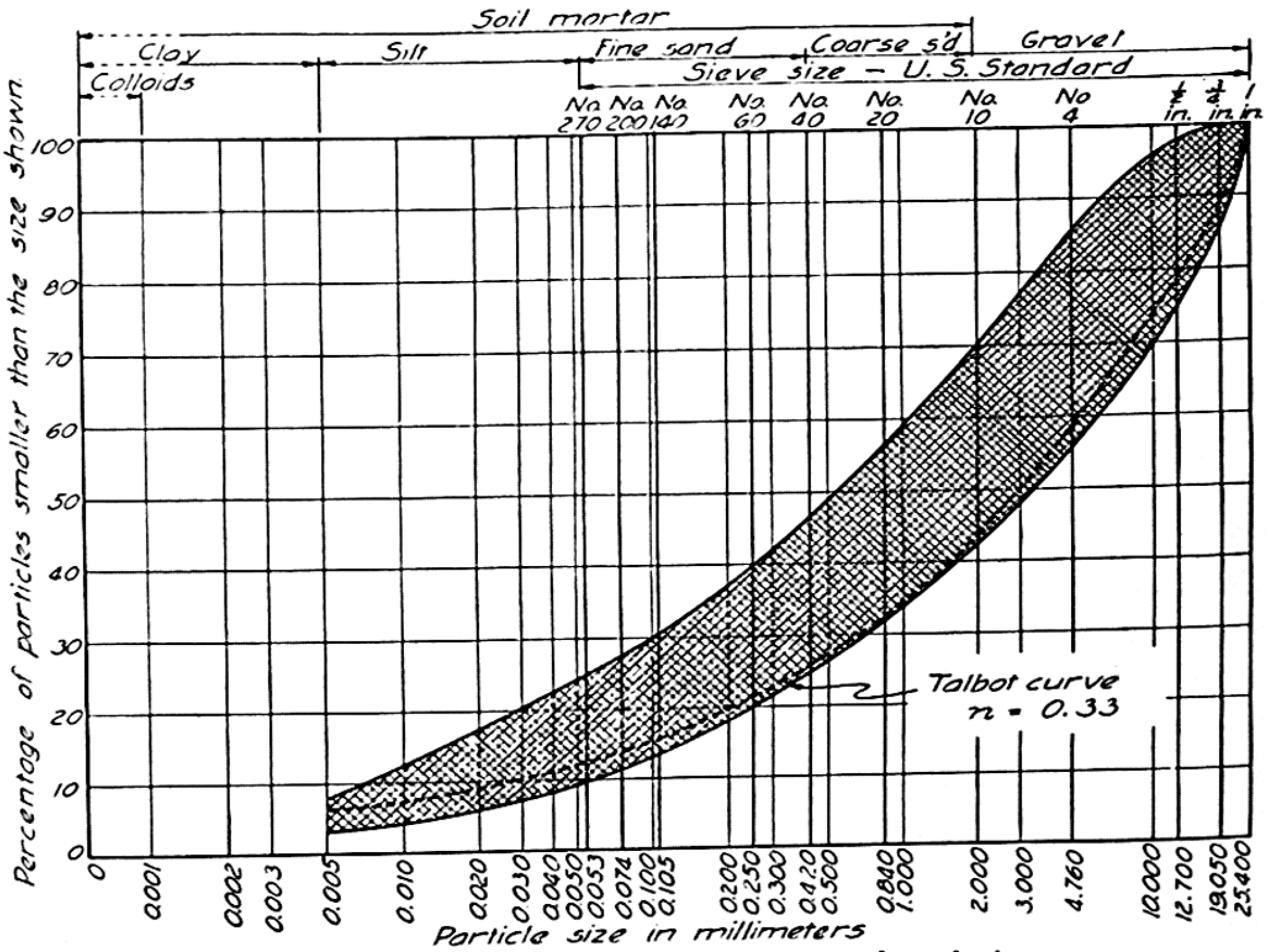


FIG. 75.—Grading limits of gravel for road surfacing.

reduced somewhat, say 20 per cent, in regions where subgrade conditions are favorable because of the character of the soil and low precipitation. Such regions are the semiarid belt of a continent, except in the stream valleys; areas of very sandy loam or deep sand; and arid regions where bituminous treatments are required to provide a bonding medium.

Many existing gravel roads have been in service so long that they are very thoroughly compacted, and the load-supporting capacity is high. Generally the gravel layer is a foot or more in thickness, and the subgrade below the gravel proper is fairly stable because of gravel that has in the past become mixed with the subgrade soil. The new gravel road is seldom built more than

8 or 10 in. thick. In the long run the best results seem to be obtained by gradually adding to the gravel layer as it becomes firm and permitting the new material to blend with the old under the pounding of the traffic. Treated or scientifically blended gravels, including those to which a deliquescent salt is added,[1] may be compacted as they are placed, by means of the sheep's-foot or a light three-wheeled roller. Each layer will be about 3 in. thick after it is rolled, and the surface will be tight and smooth and much more acceptable to traffic than the older traffic-bound types.[2]

Width of Gravel Surface.—Gravel roads with a single traffic lane are sometimes provided for local land-access highways, the surface being only 10 or 12 ft. wide. For the usual feeder road the surface is about 16 ft. wide, and for the surface on a main market road the width will be 20 to 24 ft. wide. After a few months of travel there is no longer a clear line of demarcation between the gravel and the earth shoulder. The effect of traffic and the maintenance operations is to spread the gravel out on to the shoulder and create the appearance of a surface that is wider than is actually the case.

Cross-slope.—On account of the tendency for the finer material in a gravel surface to loosen under traffic and gradually be pushed to the side of the traveled way, it is desirable to construct the surface with but little cross-slope. The prevailing practice in recent years has been to use a cross-slope of about $\frac{1}{8}$ in. per foot of width. The surface is usually kept under patrol maintenance, which insures the loose material being replaced at frequent intervals, and under that condition a low cross-slope is permissible.

Placing Gravel for Feather-edge Type.—The most common method of construction is to finish the subgrade to a level cross-section and place the gravel thereon in the quantity required to provide the width and thickness prescribed for the project and spread it by means of a blade grader to the section desired. At first the gravel will be loose and dusty in dry weather and loose

[1] "Salt-Stabilized Road Practice," *Eng. News-Record,* July 4, 1935. p. 11.

[2] BARR, JOHN H., "Stabilized-surface Methods and Costs," *Eng. News-Record,* June 27, 1935, p. 907.

"Premixed Stabilized Soil for Road Surfaces," *Eng. News-Record,* Sept. 19, 1935, p. 389.

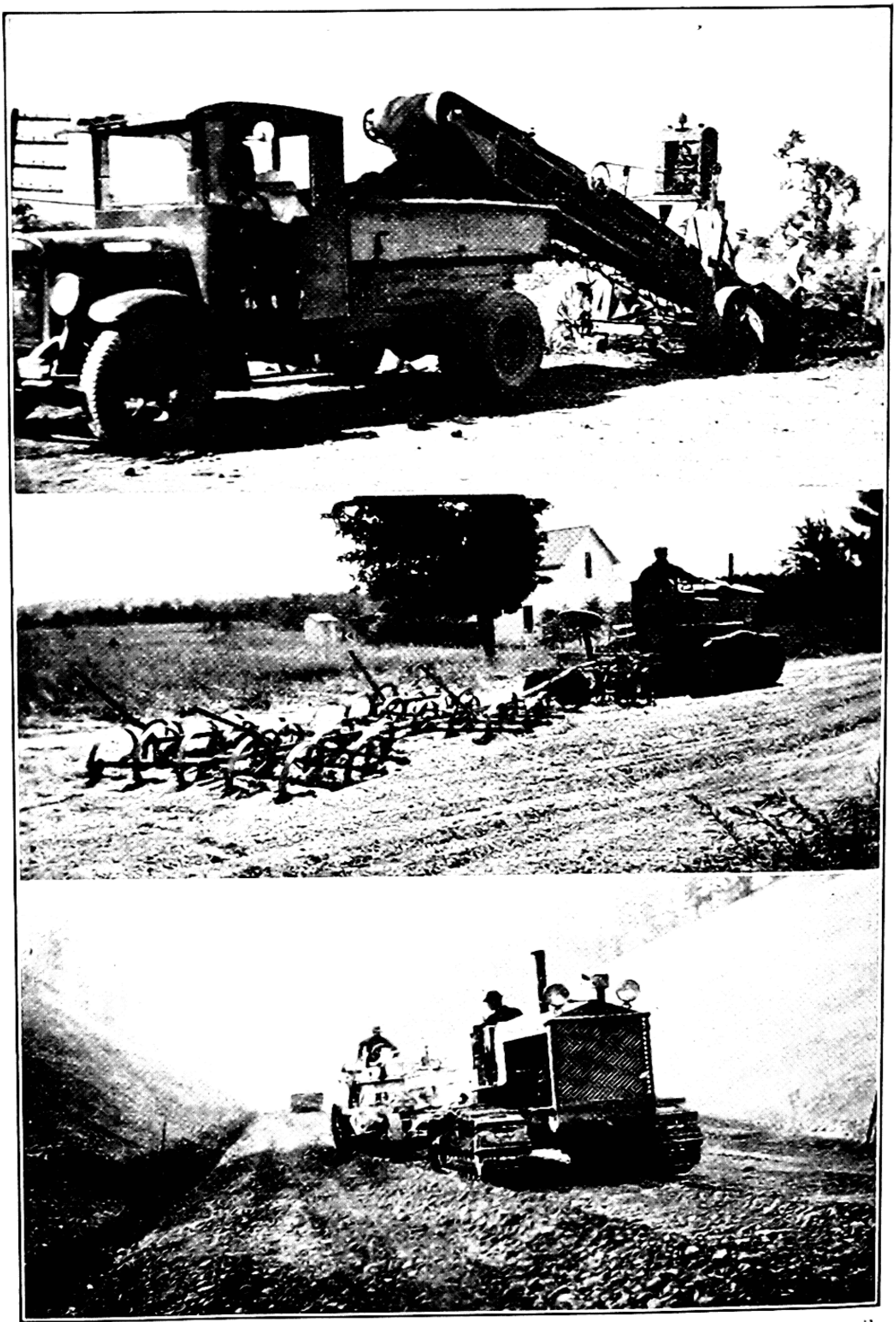


FIG. 76.—Construction operations: upper, loading topsoil; middle, mixing soil on road; lower, mixing clay and gravel with the blade grader.

and unstable in wet weather. But the rolling action of traffic will gradually compact the material. If there is insufficient fine material in the gravel, the kneading and rolling of traffic in wet weather will mix mud from the subgrade with the gravel to afford the requisite binder. It is better practice in this construction to add soil mortar to the gravels that are deficient in fine material and then wet the material and mix thoroughly before the final smoothing. The mixing can be accomplished by repeated windrowing and spreading with the blade grader. After the surface is opened to traffic, the layer is smoothed daily or at least at frequent intervals and eventually compacts into the standard gravel surface.

It has come to be recognized that by this method there cannot be a uniform distribution of the sand and soil mortar in the gravel layer and that the upper part of the layer that needs it most is likely to be deficient in binder. For the less exacting service conditions of the land-access roads the type is widely used because of its low first cost. For the more exacting conditions of the more heavily traveled roads, more precise methods of construction should be followed. In any case it is an imposition on the traveling public to use their vehicles as road rollers, and the cost in wear on tires and extra fuel is much greater than the additional cost of the rolled gravel surface.

Trench Method of Placing Gravel.—The trench method of placing gravel is used when the gravel is rolled or tamped as placed. In this method it is now the preferred practice to blend various gravels and add soil mortar when necessary to secure the particle size-distribution required for high stability.

One method of accomplishing this is to deposit the materials on the subgrade in windrows, each of which will provide for about one lane of finished surface, and then wet the materials and mix them by repeated spreading and windrowing with a blade grader. Another method employs a portable mixing machine which travels along the subgrade picking up the windrow of unmixed materials ahead of it and mixing and depositing them at the rear. Still another method is to mix the materials at a plant set up at a gravel pit or stock pile and haul the mixture to the road. In all cases the materials are carefully proportioned by blending fine and coarse gravels and adding clay to form, with the fine sand, a soil mortar of the correct consistency. Often water is added to the mixture to facilitate mixing.

The gravel is either rolled or tamped after being spread between the shoulders of the trench and wet down by means of sprinkling carts if it was mixed dry. The sheep's-foot roller is used extensively for tamping, and the light (5-ton) three-wheeled roller is also effective. When finally opened to traffic the road is smooth and firm; and although traffic will effect some additional consolidation, there is no imposition of the functions of a roller placed on the vehicles that use the road.

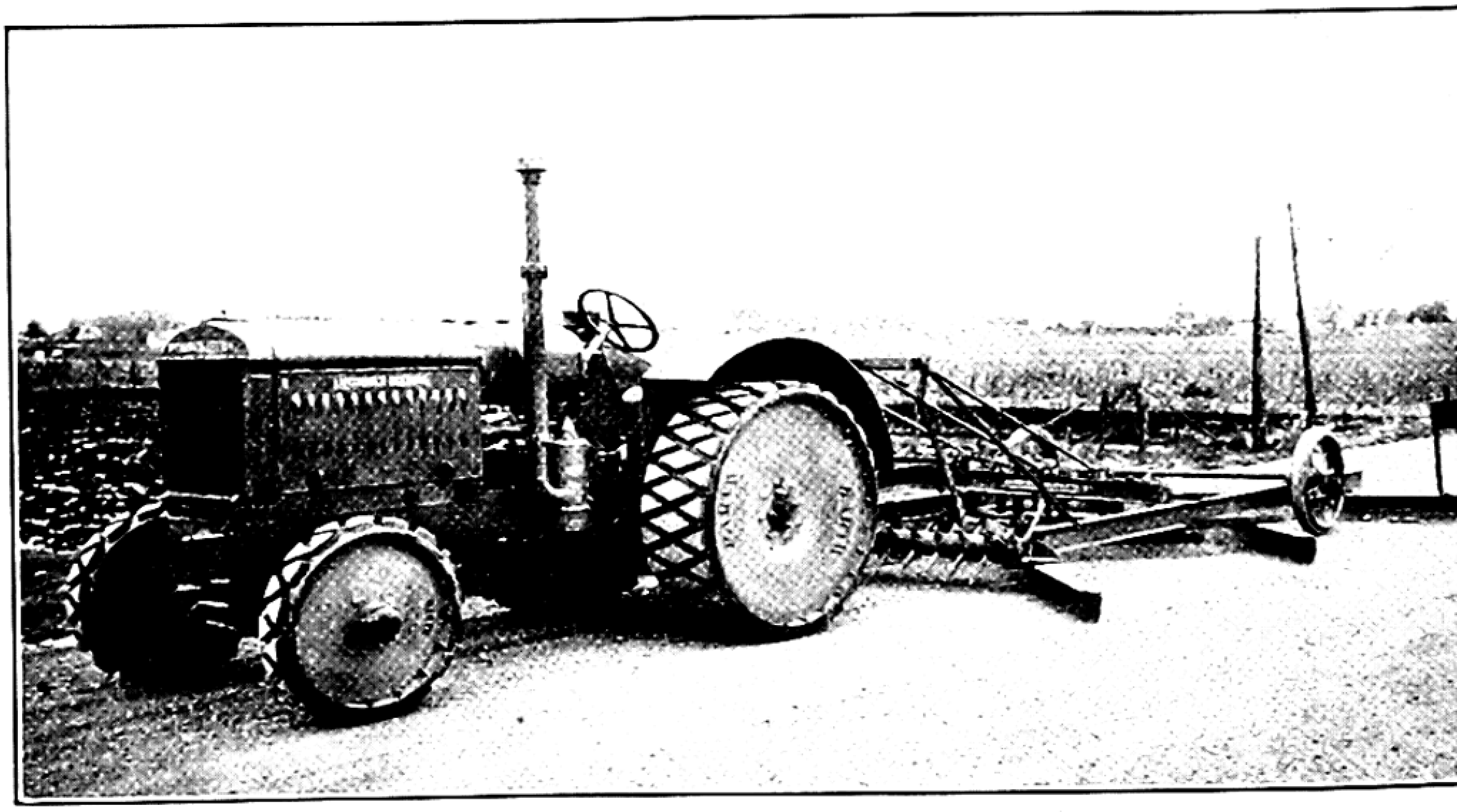


FIG. 77.—Maintainer for gravel roads.

A deliquescent salt such as calcium chloride or common salt is sometimes added to these gravel mixtures to create a condition favorable to the retention of the moisture needed for stability. The quantity of the salt needed is still a matter of investigation but on most of the projects so far completed has been in the neighborhood of ½ lb. per square yard of surface, 1 in. thick, or 18 lb. per cubic yard. This is one type of so-called "stabilized" gravel road.

Service Limitations.—The common gravel road is widely used for the light-traffic feeder roads in the highway system. Where the traffic does not exceed an average of about 300 vehicles per day and the climate is humid, this road is an economical type. In dry climates, even light traffic will loosen the gravel, and the surface will deteriorate rapidly. When the traffic exceeds 500 vehicles per day the cost of maintenance begins to mount, but —what is more significant—the surface cannot be kept in good condition except under the most favorable moisture conditions,

that is, when there is just enough moisture to keep the binding soil mortar at its most effective consistency. As a consequence of this limitation the untreated gravel road is dusty during the dry periods of the year in most climates. By daily maintenance it may be kept reasonably smooth, although even under the best care it tends to develop into transverse ridges that are so uniformly spaced that they are sometimes called *rhythmic corrugations.* By carefully proportioning the mixture of gravel and soil mortar and by compacting the layers as placed, the durability of this surface can be made adequate for traffic up to 1,000 vehicles per day, but a bituminous surface must be applied to eliminate dust.

The rather thin section constructed by the feather-edge method for light traffic is the most common adaptation of the ordinary gravel surface. The bituminous-surfaced gravel road is in a wholly different class and will be discussed in another place.

The cost ranges from $2,000 per mile for the thin feather-edge surfaces to $6,000 to $8,000 per mile for heavy-duty surfaces two lanes wide constructed of carefully blended materials placed in thin layers and rolled.

CHAPTER X

WATER-BOUND MACADAM ROADS AND PAVEMENTS

A type of road surface consisting of broken stone cemented into a solid mass by means of stone dust and water is known as *water-bound macadam.* The surface thus constructed depends for its stability upon the somewhat weak cementing property that is possessed by the dust from the rock used for the surface layer. The water-bound macadam surface that is made with good stone on a good subgrade will be sufficiently stable to carry loads of considerable weight, but the integrity of the surface depends upon the excellence of the cementing properties of the stone dust used. The water-bound macadam has little place in a modern road system to be used by self-propelled vehicles, as it will not hold together under any considerable volume of that kind of traffic. It is still constructed as the first stage in the building of surface-treated macadam or as a base course for penetration or mixed macadam. Typical cross-sections for macadam are shown in Fig. 78.

Materials.—Limestone, granite, and the various kinds of rocks designated as trap are the principal water-bound macadam materials, limestone and trap being employed much more extensively than the other kinds.

Slag, shells, burnt shale, low-grade iron ore, and sandstone are occasionally utilized but cannot be considered of wide importance as macadam materials although satisfactory for local roads in some regions.

Quality of Rock.—A macadam road is subjected to constant pounding from heavy loads, which tends to chip off fragments from the rocks or to break the rocks into smaller pieces. The quality of the rock that enables it to resist destruction in this manner is known as toughness. The toughness, as determined by the standard test,[1] should not be less than 6.

[1] Test for Toughness of Rock, A.S.T.M., Standard D3-18, 1936 B.S., II, p. 1114.

The individual pieces of stone in the surface of the road are constantly being moved slightly in the mass, as a result of distortion of the surface under heavy loads. This causes the pieces of stone to rub and grind against each other, and to resist this

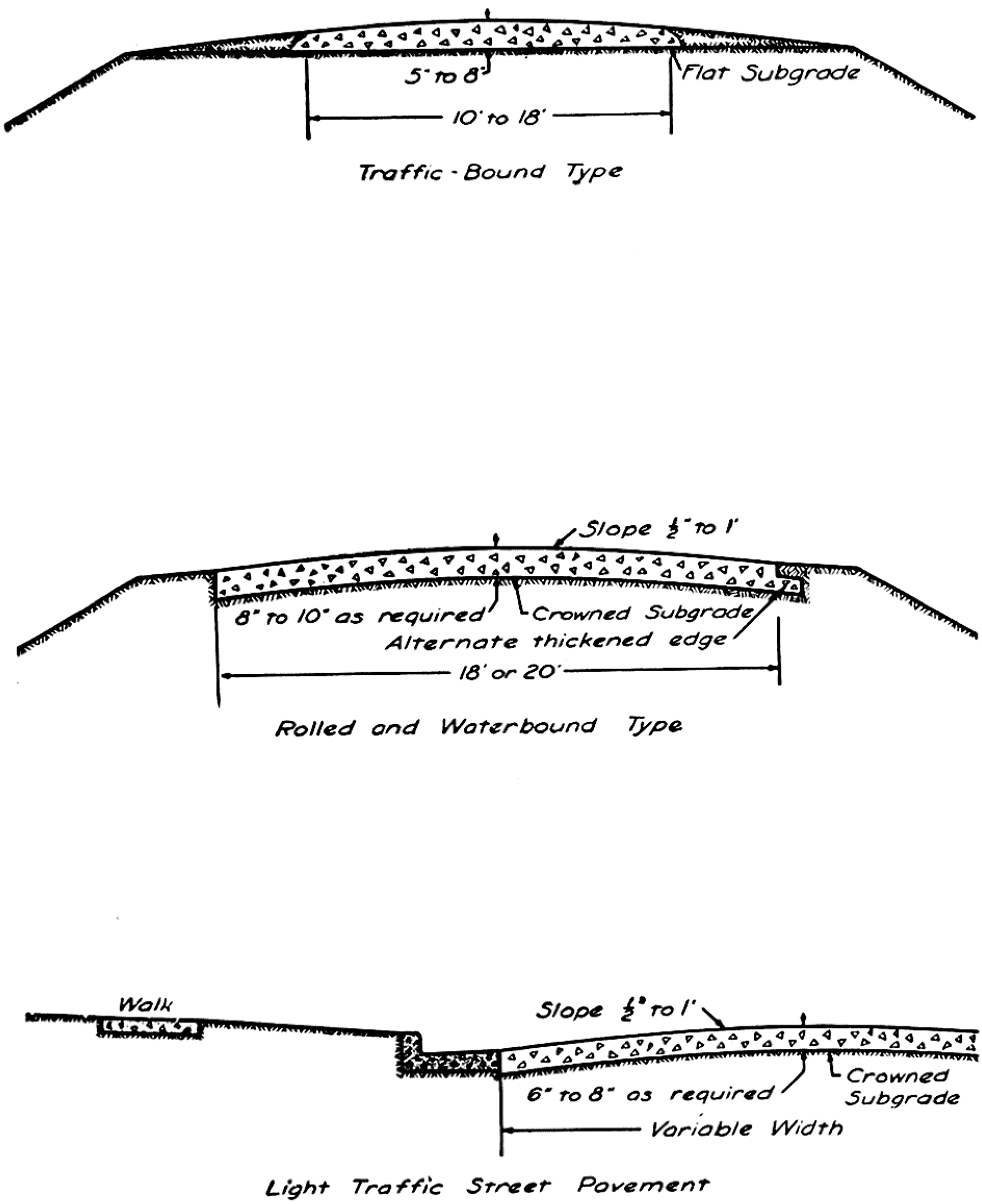


FIG. 78.—Typical cross-sections for macadam.

effect the stone must have good wearing properties. The abrasion[1] test which determines the ability of the material to resist wear is also to a degree a measure of the ability of the stone to

[1] Abrasion Test for Rock, A.S.T.M., Standard D2-33, 1936 B.S., II, p. 1040.

resist the grinding action of traffic and shock, so that the test is really a combined hardness and toughness test. The percentage of wear for heavy-duty roads should not be more than 6 but for lighter traffic might be as high as 8.

The individual pieces that make up the surface of the road are held in place partly by the mechanical interlocking induced by the rolling and partly by the cementing action of the stone dust, or screenings, which fills the interstices between the larger stones. It is therefore exceedingly important that the stone have good cementing properties. There is no recognized standard test for cementing value, and this property of a rock must be determined by field trials.

Since the surface stones are held in place by the cementing action of the screenings, and since these screenings will blow away, will be whipped off the surface and out of the spaces between the stones by automobiles, and will be washed away by storms, it is necessary for this bonding material to be replaced from some source. As the stone wears away, the dust thus made will serve to replace that which is removed, and therefore the surface will remain in better condition under mixed traffic if the stone wears fast enough to furnish the right amount of dust. Just what grade is best for any particular road will depend upon the traffic.

Size of Stone in Surface Layer.—The maximum size of stone is that which is just strong enough to carry the loads on the road without crushing. A tough stone need not, therefore, be so large as one low in this property. The size is further limited, however, when in order to secure a stone large enough to withstand traffic, a size is reached where the stone will tip in the surface as a wheel rolls over it. Although some engineers permit the use of a stone passing a 3½-in. ring, it is probable that a size passing a 2½-in. ring is about the largest that can be recommended. If the stone is high in the property of toughness, the size need not be so great.

As a general principle it may be said that if the French coefficient of wear exceeds 12, the size for the surface layer may be from 2½ down to ¼ in., and the size from ¼ in. down may be utilized as screenings for bonding the surface. If the stone has a coefficient of wear less than 12, the size may be from 3½ down to about ¾ in., and the screenings from ¾ in. down.[1]

[1] See also Standard Specifications for Broken Stone for Water-bound Macadam Surface Course, A.S.T.M., 1936 B.S., II, p. 1038.

Size of Stone for Lower Course.—It is not so important to have the stone for the lower course of any particular size as long as it is convenient to place and roll. Good serviceable roads have been built with the lower course made from screenings ¾ in. down, but this cannot be recommended as good practice. Any size obtainable up to that which passes a 4½-in. ring will serve. The lower course is generally made of a stone larger in size than that which composes the upper course because of the economy in cost of crushing. Not infrequently, however, the same size of stone is used for upper and lower courses.

Size of Stone for Telford Foundation.—The requirements as to the permissible size of stone for Telford foundation are not very rigid. A size that can readily be handled by one man is suitable. One dimension ought to be within 1 in. of the specified thickness of the foundation which may be 6 to 8 in.; the width as set is from 5 in. to 1 ft.; and the length from 8 to 15 in. The essential requirement is that the pieces conveniently lie up to the required thickness of course. If too large, the stone will not lock together so as to be stable under rolling. The larger pieces are "chinked" with spalls so as to hold them firmly in place.

Crusher-run Stone.—Stone is sometimes taken directly from the crusher and placed on a road. Since the screenings are mixed with the stone, it may be compacted by rolling or by traffic and will bond into a fairly stable surface. Such a method of construction is suitable only for light-traffic roads, and the surface is not likely to wear evenly. Since rubber tires do not constitute a very good roller, it is best to compact the stone with the macadam roller.

Nature of the Macadam Surface.—It will readily be seen that a macadam surface cannot distribute the load it carries over a very great area of subgrade because the layer of bonded stone does not possess flexural strength. It is commonly assumed that the load is carried to the foundation on 45-deg. lines from the area of application of the wheel or other load. Whether or not this is true, it has been shown by many examples that ordinary soil, when well drained, will carry wheel loads up to 2 tons if the macadam is made 8 in. thick. Exceptions must be made of those soils which are of seepy or peaty nature and are impossible of effective drainage. Here the Telford type of foundation would be used.

Subgrade for Stone.—The subgrade really carries the load imposed on the road surface by traffic units on any kind of surfaced road; and with the gravel and macadam types it is especially necessary to secure as stable a subgrade as is possible within the limits of funds and available materials. The macadam type is used only for light- or medium-traffic densities and individual wheel loads up to about 4,000 pounds. Generally sand or gravel cannot be obtained readily for mixing with subgrade soil to give it stability, and resort must be had to careful drainage and the removal from the subgrade of pockets of poorly graded and unstable soils. The area to be covered by the macadam may be constructed along the lines of the densified treated earth road if materials are available. In any case the subgrade is rolled thoroughly before the macadam is placed.

Thickness of Macadam.—The thickness of the completed macadam road is variously specified in regions where this type is still built but will generally lie between 8 and 10 in. if the roadbed soil is reasonably stable. It appears that the 8-in. thickness predominates. If the foundation is of gravel, deep sand, or other equally stable material, the thickness may be as little as 6 in. It is not probable that a macadam surface will prove to be economical, if a thickness greater than 12 in. is necessary for stability. The thickness mentioned is in each case that which is obtained after thorough rolling which is about 80 per cent of the thickness of the loose material.

A roller will not compress properly more than about 6 in. of loose stone; therefore it is common practice to place the material in two approximately equal layers, rolling the lower course thoroughly before the upper is spread.

When the Telford type of foundation course is employed, its thickness is usually about 6 in. but may be as great as 8 in. The Telford base is carefully placed by hand, and all chinks between the stone filled with spalls, gravel, or crushed stone and then rolled with a 15-ton roller.

The upper layer for a Telford macadam is about 3 in. thick after it has been rolled.

Quantity of Stone Required.—A macadam surface 8 in. thick, after rolling, requires 27 cu. yd. of stone per 100 ft. of road 9 ft. wide, to which must be added 3 cu. yd. of screening if the upper course only is bonded and 5 cu. yd. if both courses

are bonded. Pavements of other widths and thicknesses may be computed on a basis of a shrinkage of 20 per cent in the layer of stone during rolling. The quantity of screenings required to bond a layer of stone is 20 per cent of the volume of the layer. These quantities hold true for most materials within the limits required for estimates but will inevitably vary slightly with the size of stone, fineness of screenings, and actual density of the layer after rolling.

Roadbed and Shoulders.—It is imperative that the broken stone be placed between substantial berms or shoulders of earth so that when rolled it will be compacted, not merely spread out. The earth shoulders may be formed by removing the earth from the middle of the road for the required width and grading it out to the sides, thus forming a trench for the stone, and this method is followed when the road to be surfaced is already well shaped up and has the requisite cross-slope.

If the road to be improved lacks the cross-slope or crown necessary for good drainage, the shoulders are formed by drawing material from the sides of the road and the ditches. The shoulders thus obtained will be loose and will require thorough rolling prior to placing the stone to insure that they will not spread out when the stone is rolled.

It not infrequently happens that both methods of forming shoulders will be necessary on adjacent sections of a road, on account of the lack of uniformity of the existing earth-road section.

The surface upon which the stone is placed is referred to as the subgrade and usually is made with a convex cross-section. This is desirable for any kind of hard surface that is sufficiently porous so that there is a possibility that some water may soak through to the subgrade. When the subgrade is crowned the water will work to the edge and soak away into the shoulder or be conducted away through broken-stone drains.

The subgrade must be shaped with care so that no uneven places exist because these will either reduce or increase the thickness of the macadam depending upon whether they are above or below grade.

Since the stability of the surface depends to a large measure upon the solidity of the subgrade, it is rolled thoroughly. If soft and yielding places are encountered, the soft material is removed, and good material substituted; or if ground water is

encountered, the necessary underdrainage is put in to remove the water and permit the proper compacting of the earth.

Placing the Stone.—If the Telford base is employed, it is placed and rolled as previously described. The stone is hauled in trucks equipped to spread it to any desired thickness as dumped, or it is dumped into a spreader drawn along the subgrade by the truck. After the spreading, a small amount of smoothing by means of a blade grader will prepare the layer for rolling.

FIG. 79.—A poorly constructed macadam which is raveling.

Blind Drains.—If there is rain during the construction period the water will percolate through the layer of stone and collect in the subgrade, particularly along the shoulders. To protect against damages at this time, blind drains, which are merely trenches sufficiently deep to permit the water to run freely from the subgrade, are dug and partially filled with coarse broken stone which is covered with earth to the level of the shoulder. The stone is generally put in them as the lower course is deposited. These drains are spaced about 50 ft. apart on each side of the road but may be closer together in the low places in the road and omitted near the hilltops. On long hills it is desirable to excavate every third pair of the lateral drains to a depth of about 5 in. below the subgrade and extend them to meet at the middle of the road. The drains are sloped slightly downhill on grades and are at right angles to the center line elsewhere.

Rolling.—For limestones a roller weighing about 400 lb. per inch of width of roll is used; and for trap and similar stones, one weighing about 600 lb. per inch of width. On the lower course the rolling begins at the edge of the stone, and the roller moves parallel to the edge of the stone and at a speed of about 100 ft. per minute. The machine moves backward and forward, edging in toward the middle at each trip; and when the middle is reached the roller is taken to the opposite side of the road, and the process repeated. The roller then returns to the side first rolled, and this is repeated until the stone is thoroughly compacted. A roller will properly compact and bond about 50 sq. yd. of road surface per hour.

When rolling is completed each piece of stone will be wedged tightly between its fellows and will thus be restrained against any lateral displacement. At this stage of the rolling it will be noted that the stones have begun to break under the roller. If the rolling is continued the stone will crush and wear against each other and begin to loosen in the surface.

Rolling the upper course of the macadam road is carried out in the same manner as on the lower course except that the rolling begins out on the shoulder about 4 ft. To do this successfully the shoulders must be trimmed to such a height that they will roll down level with the stone.

During the rolling, some uneven places will appear, and these are brought up by the addition of a small quantity of stone of the same size as is used in the course. A skillful roller operator will frequently sight along the surface of the macadam to detect uneven places and will manipulate the roller to produce a smooth, uniform, tightly keyed surface. Much depends upon the skill and experience of the roller operator, and thorough rolling is a vital part of the construction of macadam roads.

Applying Screenings.—After the rolling on the upper course is completed no vehicles are permitted to use it until the screenings have been applied. These are dumped in piles at the side of the macadam and are thrown on to the surface with a sweeping motion of the shovel so that they will not be deposited in piles. The quantity used is that which will just cover the stones. Frequently the screenings are brushed into the surface with fiber brooms. Some engineers then sprinkle the surface lightly, and others roll the screenings dry. As the rolling proceeds, bare places will appear, and screenings are added until the layer is filled and a small quantity remains on the surface.

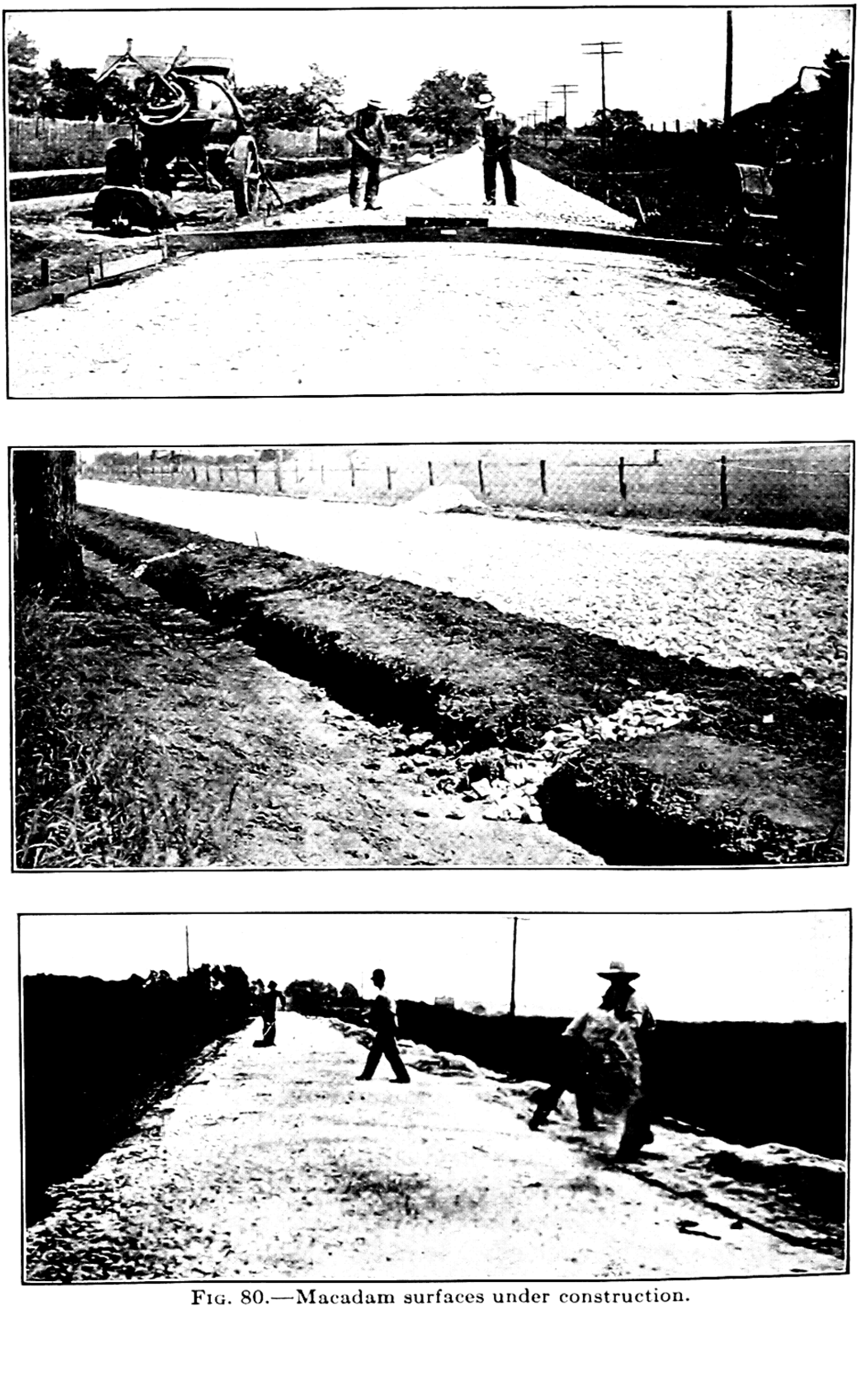


FIG. 80.—Macadam surfaces under construction.

Puddling.—When the required quantity of screenings has been rolled into the layer of stone, the surface is sprinkled and rolled, the roller following immediately behind the sprinkler so that the spray of water falls on or just ahead of the roller. As successive trips are made with the roller, the dust and the water form a mortar which is worked into every crevice in the surface. This operation of *puddling* is exceedingly important if a durable surface is to be constructed.

When puddling is completed the road is closed to all traffic until the screenings set up, which may be for 2 days or a week, depending upon the weather. If it is very hot and dry, it is well to sprinkle daily for 2 or 3 days.

After the screenings have set, the surface may be rolled again after each rain, and the more rolling it gets at such times the better.

Finishing the Shoulders.—The shoulders should be neatly trimmed to the proper cross-slope, the back slopes to the ditches and the side slopes on cuts and fills trimmed, and the ditches properly shaped and cleaned of all loose earth.

Maintenance.—The surface may suffer almost entirely from one of the following causes, or it may be subjected to all of them simultaneously. The method of maintenance will depend upon just how the surface wears.

If motor traffic predominates, loss of binder will result, and in order to maintain the surface under such conditions, it will be necessary to renew the binder as fast as it is swept off. Screening or bonding gravel may be used for this purpose and where possible should be put on while the road is wet. If the volume of motor traffic is in excess of 200 vehicles per day, bituminous treatments are required for satisfactory maintenance.

If the surface wears rapidly, new materials must be added from time to time to maintain the thickness. Chuck holes and ruts will appear, and these must be filled to maintain the smooth surface.

When new stone is added, the old surface must be loosened to insure that the new surface will unite with the old. Where the area to be patched is small, the loosening is done by hand with picks; but when extensive resurfacing is necessary, the scarifier or spikes in the roller wheel can be most economically employed. The surface is loosened to a depth of about 4 in.; the new material is added and rolled or tamped in place. Screenings are then

spread, and the patches of new surface bonded, just as in the construction of a new road.

It is important to keep a broken-stone road in a smooth condition, because if a chuck hole starts or a rut appears it will rapidly increase in size, and the comfort and convenience of traffic will be interfered with.

No matter how faithfully a water-bound macadam road is maintained, it will be inadequate to meet the demands of motor traffic unless treated with a bituminous binder to keep the surface intact.

Costs.—The cost of light traffic-bound macadam surfaces ranges between $3,000 and $7,000 per mile 18 ft. wide. The rolled and water-bound macadams range in cost between $6,000 and $15,000 per mile for surfaces 20 ft. wide. The cost of maintenance averages about $750 per mile per year exclusive of bituminous treatments.

TOOLS AND MACHINERY USED FOR MACADAM CONSTRUCTION

Earthworking Tools.—The earthwork incident to hard surfacing is performed with the tools and machinery that have already been described in the chapter on earth-road construction.

In addition, many other types of machinery are needed.

Motor Trucks.—The motor truck has almost entirely superseded all other methods of hauling road materials, particularly where the hauling is over reasonably good roads or for delivery over pavements.

Motor trucks are made with capacities up to 5 cu. yd., and motor trucks of the trailer type are of even greater capacity. These trucks are self-dumping, travel rapidly, and when used in connection with some quick-loading device will handle large quantities of materials, spreading the material as it is dumped.

Loading Devices for Materials.—The locomotive crane with clamshell bucket used in connection with the storage bin or loading directly into the trucks is used almost exclusively for unloading broken stone from cars.

Self-propelled Rollers.—Steam- and gasoline-driven rollers are quite general in macadam road construction and in various kinds of pavement construction. Two types, known respectively as *macadam rollers* and *tandem rollers*, are built. The macadam, or three-wheeled, roller is designed for compacting embankments, for rolling the foundation of roads and pavements, and for

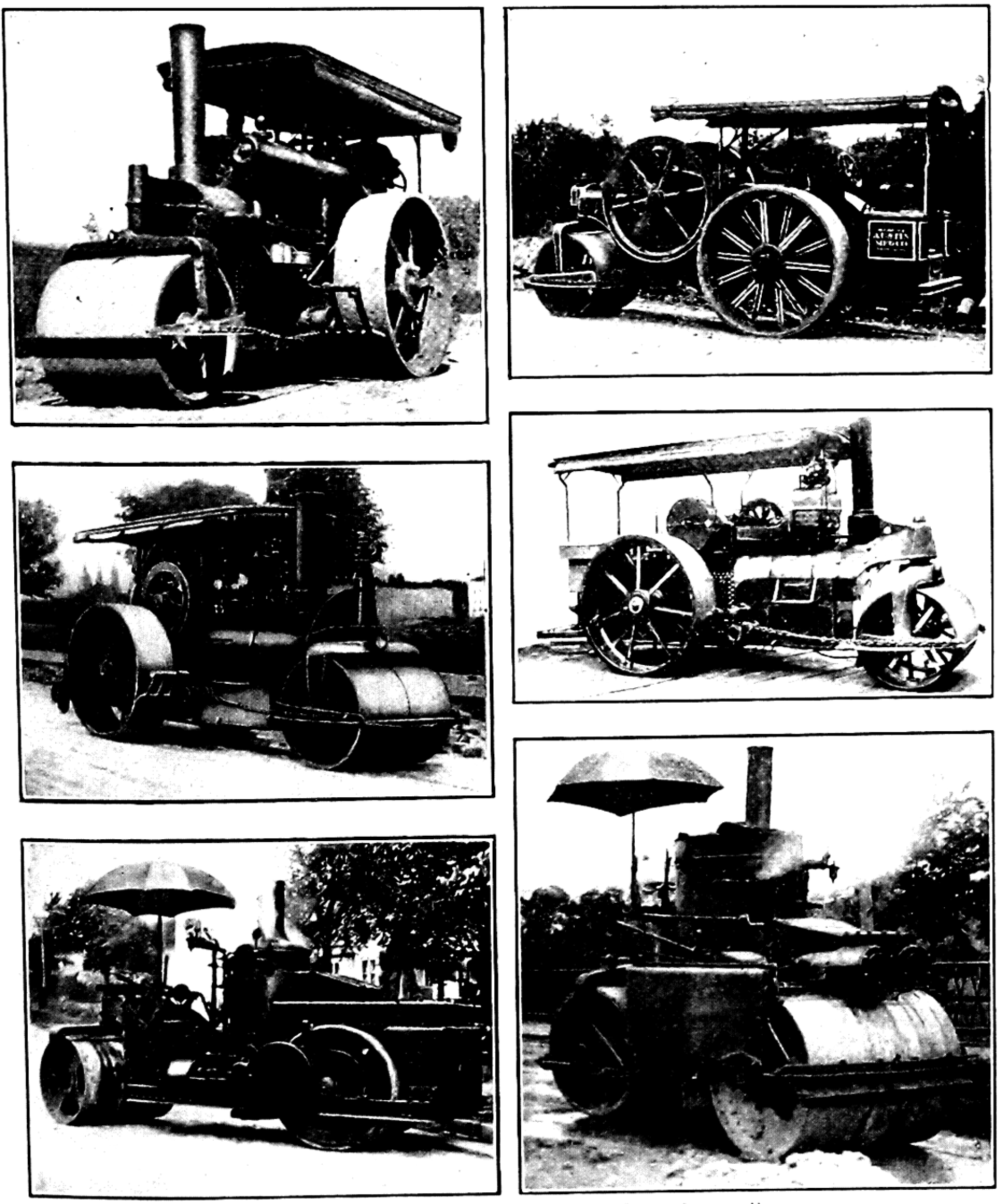


FIG. 81.—Some types of macadam and tandem rollers.

the construction of the various kinds of macadam roads. The weight may be from 8 to 20 tons, but for all-round work the 10-ton size is most suitable. The width of the roller varies with the weight, but the relative amount of weight on the front and rear rolls should be about the same for all sizes; and since the rear roll is larger in diameter than the front, the weight on the rear roll should be about six-tenths of the total weight. The combined width of the two rear rolls should equal the width of the front roll, and the path of the rear roll should overlap that of the front roll about 4 in.

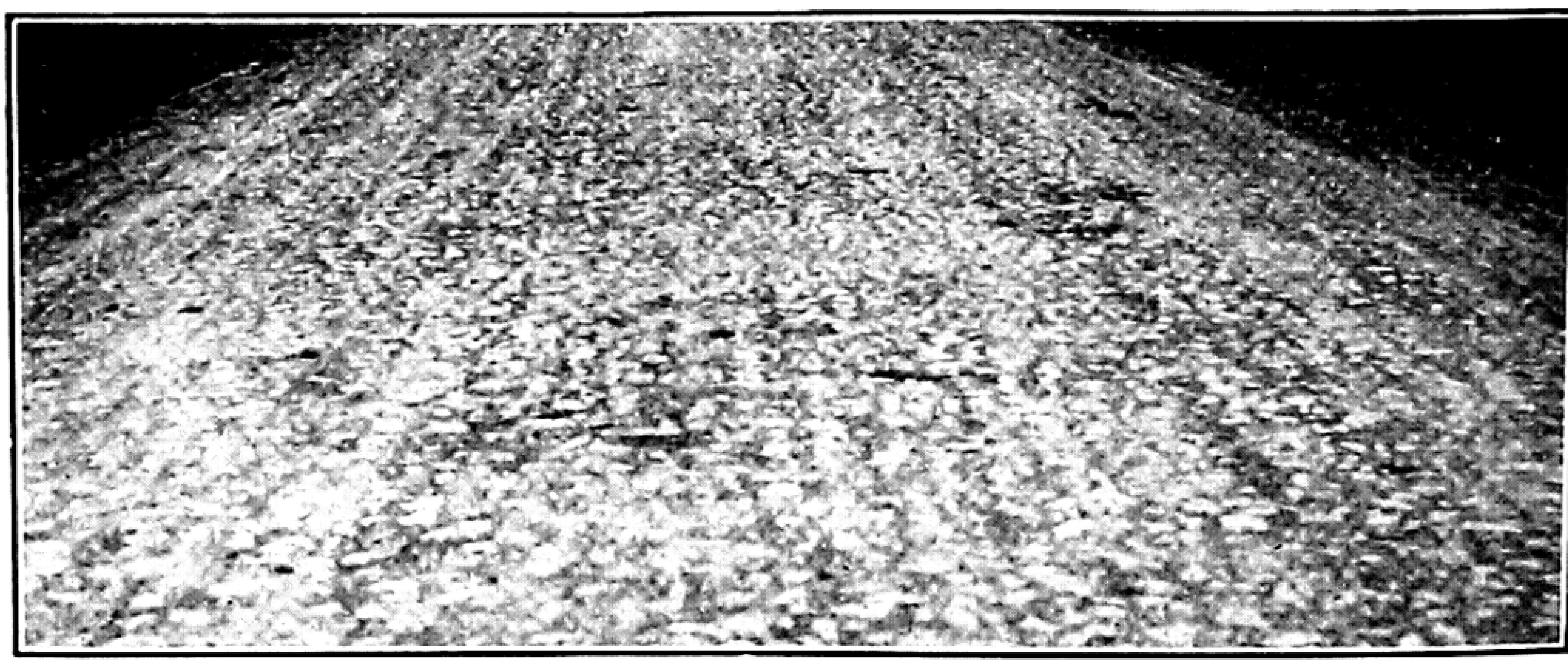


Fig. 82.—A macadam road in need of surfacing.

Scarifiers.—The scarifier is designed for loosening the surface of gravel or macadam roads when repairs are to be made. It consists of two or more hard-steel teeth set in a heavy frame so arranged that the depth to which the teeth penetrate the surface of the road can be adjusted. The scarifier is usually drawn by a tractor or roller, and some types are built so that they can be steered independent of the tractor. Scarifier attachments are also provided for some of the heavier blade graders.

Portable Stone-crushing Plants.—For portable crushing outfits the jaw type of crusher is most commonly employed, although the gyratory crusher is also made in small sizes. The crushing plant consists of the crusher, which will have a capacity of about 10 cu. yd. per hour, and the screening equipment and storage bins. The cylindrical revolving screen is used and is equipped with sections of screen to suit the work in hand. Storage bins are provided with sufficient capacity to take the output between loads so that the plant can run regardless of the regularity of

hauling. For portable outfits the ordinary well-driller's rig is used for drilling the ledge for blasting out the stone.

Stationary Crushing Plants.—For the larger installations the gyratory type of crusher is employed, and the other machinery and method of operation are much the same as for the smaller plants, except as regards size. The drilling is done by means of steam or air drills.

All stone crushers are built in such a way that the maximum size of the product can be adjusted, and the screen sections can be adjusted by removing the perforated metal and replacing with another size.

EXAMPLES OF GOOD PRACTICE

Wisconsin[1]

Surfacing—Quantities Required

The following table gives the number of cubic yards of material per mile to make a given loose depth for various widths of roads:

Table XX.—Quantity of Material Required for Macadam Roads

Depth of loose material in inches	Width of surfacing				
	9 ft.	14 ft.	15 ft.	16 ft.	18 ft.
	Cu. yd.	Cu. yd.	Cu. yd.	Cu. yd.	Cu. yd.
1¼ in. (screenings)	180	280	300	325	367
3 in.	440	684	733	782	880
4 in.	587	913	979	1,043	1,174
5 in.	734	1,141	1,222	1,304	1,468
6 in.	880	1,369	1,466	1,565	1,760
Square yards of surface per mile	5,280	8,213	8,800	9,387	10,560

[1] *Bull. 4, Wisconsin Highway Comm.*

The following table gives the number of linear feet of 9-ft. road that a load of a given size should cover for various loose depths.

Foremen must compel spreaders to use this table.

TABLE XXI.—SHOWING LINEAR FEET OF 9-FT. ROAD FOR VARIOUS SIZES OF LOAD

Weight of load		Loose depth in inches				
Granite, lb.	Limestone, lb.	Size of load, cu. yd.	3 in.	4 in.	5 in.	6 in.
2,800	2,500	1	12 ft.	9.0 ft.	7.2 ft.	6.0 ft.
3,500	3,125	1¼	15 ft.	11.25 ft.	9.0 ft.	7.5 ft.
4,200	3,750	1½	18 ft.	13.5 ft.	10.8 ft.	9.0 ft.
4,900	4,375	1¾	21 ft.	15.75 ft.	12.6 ft.	10.5 ft.
5,600	5,000	2	24 ft.	18.0 ft.	14.4 ft.	12.0 ft.
6,300	5,625	2¼	27 ft.	20.25 ft.	16.2 ft.	13.5 ft.
7,000	6,250	2½	30 ft.	22.5 ft.	18.0 ft.	15.0 ft.
7,700	6,875	2¾	33 ft.	24.75 ft.	19.8 ft.	16.5 ft.
8,400	7,500	3	36 ft.	27.0 ft.	21.6 ft.	18.0 ft.

CHAPTER XI

ROAD SLABS OF CONCRETE

The concrete road slab is used as a foundation for a sheet or block wearing surface or as a combined foundation and wearing course. This chapter deals primarily with the concrete pavement slab but includes a brief discussion of concrete foundations.

The Function of the Slab.—The concrete road slab, whether it is to be used as a foundation course or as a wearing course, has the property of an imperfectly elastic solid, and hence on any

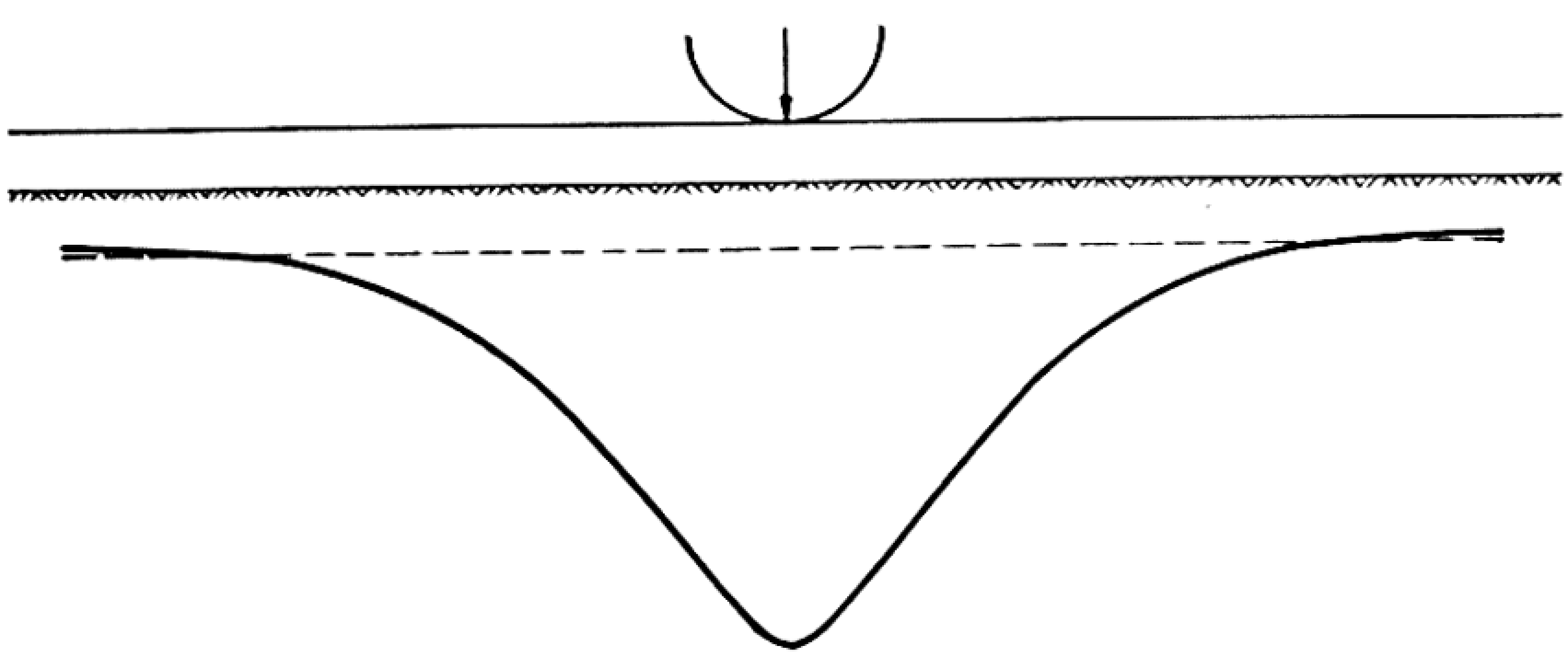


FIG. 83.—Illustrating curvature of road slabs under wheel loads.

subgrade its stability under load is related to its structural strength. The use of slabs of this type introduces illusive problems of structural design, since consideration must be given not only to the traffic load to which the slab is subjected but also to temperature and moisture effects on the concrete and the influence of the subgrade support upon the stability of the overlying slab.

The nature and magnitude of the stresses to which road slabs of concrete are subjected by traffic and by climatic influences have been under intensive investigation for at least 20 years; and although many baffling questions have been encountered, a great deal has been learned that is of assistance to the engineer in the design of concrete road slabs. It is an axiom of highway design that, in the last analysis, the traffic loads are carried by

the soil[1] which constitutes the subgrade upon which the road slab is constructed. The behavior of the road slab under loads and climatic changes is influenced to a considerable extent by the fact that it rests upon the natural soil, with which the slab does not maintain intimate contact under all conditions of loading and of temperature distortion.

LOADS ON ROAD SLABS

Definitions Relating to Wheel Loads.—In the discussion of loads on road slabs certain conventional terms will be employed in accordance with the following definitions.

Wheel Load.—When a vehicle is standing upon a horizontal surface with a wheel upon an accurate weighing device, the weight indicated is the wheel load of that vehicle. It will be noted that the wheel load includes a portion of the weight of the vehicle itself and a part of the live, or cargo, load that the vehicle is carrying.

Static Wheel Load.—The wheel load of a vehicle that is standing on the roadway surface.

Impact Load.—The force applied to the road surface by a vehicle wheel subjected to vertical acceleration in such a manner as to cause a vertical reaction against the road slab. For example, the force applied to the road slab when a wheel rolls on to a high joint in the slab and again when it rolls off.

Dynamic Load.—The dynamic load is the one produced by a rapidly moving wheel load including the effect of the sudden application of load and actual impact load.

Equivalent Static Load.—The static load that will produce a stress in a slab equal to that produced by a given dynamic load.

Edge Load.—A load applied at the marginal edge of a road slab, as at *D*, Fig. 84.

Corner Load.—A load applied at the corner formed by the intersection of transverse cracks or joints with the longitudinal joint or with the edge of the slab, as at *P*, *B*, and *C*, Fig. 84.

Interior Load.—A load applied on the slab at a distance of at least 3 ft. from a crack, joint, or edge, as at *E*, Fig. 84.

Edge Stress, Corner Stress, Interior Stress.—The stresses corresponding to the respective loads as defined above.

[1] "Researches on Structural Design of Highways," *Trans. A.S.C.E.*, Vol. 88, pp. 264–305, 1925.

Impact Reaction.—The vertical reaction of the road slab under an impact load.

Loads on Road Slabs.—The load that a road slab must carry is produced by vehicles that may be moving or standing.

Area of Application of Load.—The load is applied to the slab or the wearing surface supported by the slab, at the area of contact between the tire of the vehicle and the road surface; and in highway design it may be considered that the tires are of the pneumatic type. The area of contact between the tire and the road

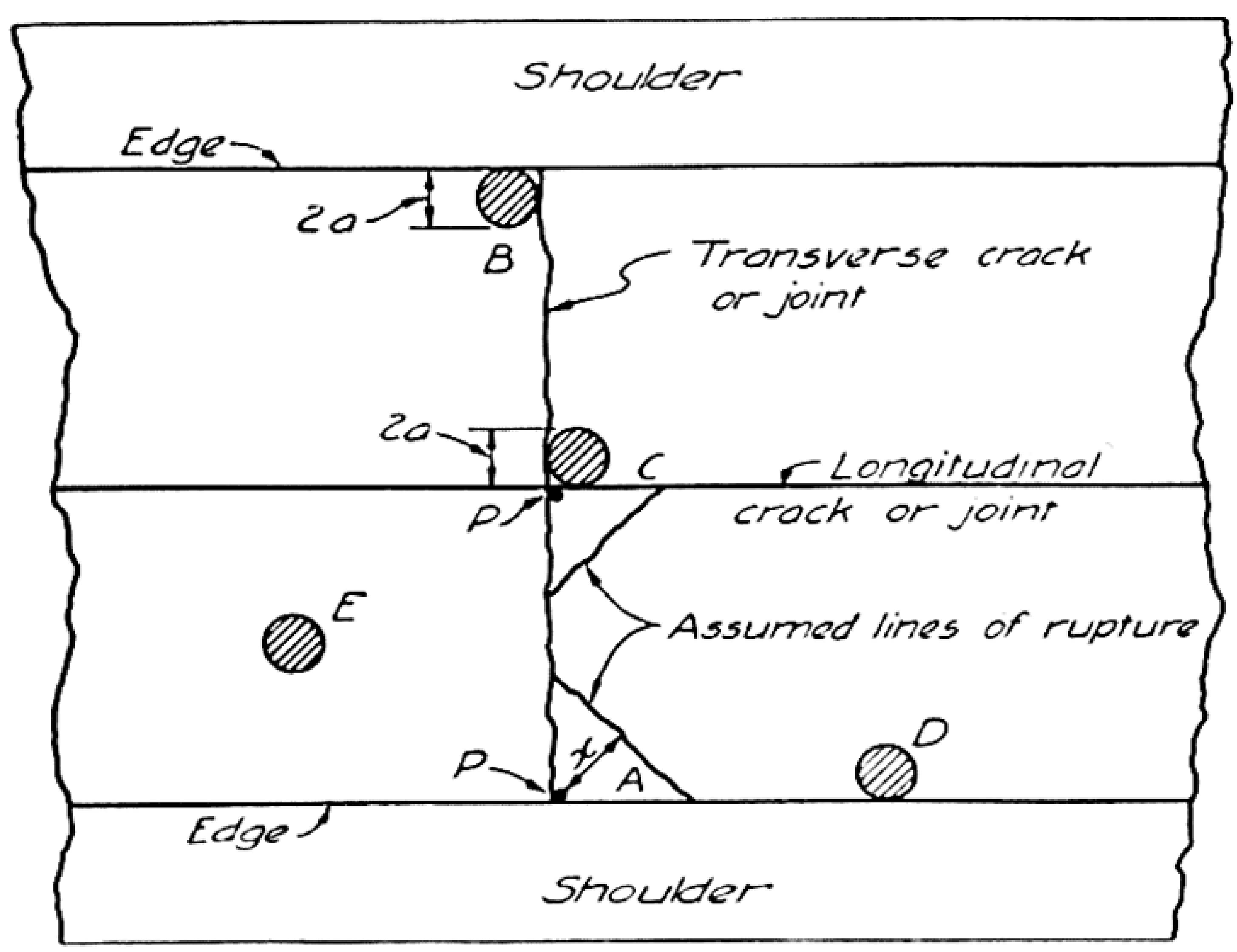


Fig. 84.—Illustrating a road slab with characteristic cracks.

surface varies in shape and size with the type of tire equipment, and for the purposes of road design it is necessary to assume that the load is uniformly distributed to the road surface over an area equivalent to the actual area of contact between the tire and road surface. No conclusive data are available that show the correct values of a for use in design formulas like (10), but for the critical loads on dual tires $a = 8$ in. is probably not far out of line. For the maximum loads on single tires $a = 6$ in. may be used until better data are available.[1]

[1] Spangler, M. G., "Effect on Slab Stress of Area of Tire Contact," *Eng. News-Record*, Vol. 112, p. 831, June 28, 1934.

Bradbury, Royall D., "Reinforced Concrete Pavements," Wire Reinforcement Institute, Washington, D. C., p. 18, 1938.

Teller, L. W., and J. W. Buchanan, "Pressure over the Contact Area of Tires," *Public Roads*, Vol. 18, No. 11, p. 195, December, 1937.

Magnitude of Loads.—For the purposes of design it is necessary to know the magnitude and frequency of application of the *maximum* load to which a road slab is subjected. The character of the traffic on a route proposed for improvement can be determined quickly and at modest cost by a short-period traffic census (page 449), and the design should be based on the maximum loads that may be expected with some frequency, such as ten times per day. It will seldom be necessary to provide against the very heavy load that uses the road once or twice a month. Generally it is an overload that can be eliminated by enforcement of vehicle weight limit laws. The maximum load on trunk highways is likely to be in excess of the maximum wheel load permitted by the laws of the states. As a matter of fact there is no geographical limitation to the operation of the modern freight motor vehicle or passenger bus, and consequently there is a tendency for the heaviest loads permitted in any state to be encountered in other states, as it is impossible to enforce state line barriers to maximum wheel loads permitted in other states. The traffic laws in effect in 1939 in several states permit a gross load on four wheels of 32,000 lb. and a maximum wheel load of 9,000 lb. There are various restrictions as to the type of tire and load permissible when the road is not in its best condition.

On the basis of the present laws the designer must assume that the maximum static wheel load will be as great as 8,000 lb. on heavy-duty rural highways and municipal thoroughfares, and an improvement on any street or highway may attract traffic that will subject the pavement to the maximum wheel load permitted by law, plus some overload at times and plus any dynamic load produced by impact.

County roads may be divided into two classes, one of which could be thought of as the main county highways, and these are occasionally subjected to wheel loads up to 5,000 lb. or more. The remainder of the county system is unlikely to carry wheel loads in excess of 3,000 or 4,000 lb.

City streets in strictly residential area will carry few wheel loads in excess of 3,000 lb. Arterial streets and those in warehouse and wholesale districts will generally be subjected to the legal maximum wheel loads and perhaps some loads considerably in excess of that because of the necessity of moving construction materials over the city streets. Certain streets in the cities are

restricted to passenger vehicle traffic, and are therefore easily classified.

The first step in the design of a road slab for a known location is to ascertain the probable maximum wheel load that the slab will be called upon to carry. That requires an analysis of the traffic pattern in the area and a forecast of what may develop in the future in the light of known plans for the improvement of the street or road system.

Impact.—The extent to which the stress that would be induced in a road slab by a static wheel load is increased when the same wheel load is applied suddenly and under conditions that may cause impact, has been thoroughly investigated and it is quite apparent that the conditions that induce impact[1] can develop on any highway. It has been shown that vehicles with pneumatic tires operating on *smooth* surfaces produce dynamic loads but little larger than the static wheel load. That is to say, the amount of impact is negligible. It has also been shown that unevenness in the road surface due to artificial obstructions, which may be neglected as a factor in design, or unevenness that develops in the road surface through the action of the elements and wear may cause vehicles with pneumatic tires to produce impact of considerable magnitude. The dynamic load of vehicles with pneumatic tires on a pavement that has several years of use even though it is kept in repair may reach 1.3 times the static wheel load. On pavements that are not in good repair and have become quite rough, from frost action or the pounding of traffic, this reaction may be double the static load.

Impact Stresses.—In the researches to determine the magnitude of the impact factor on highway slabs, it has been shown[2] that the stress produced in the pavement slabs by a dynamic load (page 268) of any magnitude is somewhat less than the stress produced by a static wheel load of the same magnitude. This is explained in part by the yielding of the slab under the blow and in part by the slow absorption of the load by the subgrade soil

[1] Buchanan, James A., and J. W. Reid, "Motor Truck Impact," *Public Roads*, Vol. 7, No. 4, p. 69, June, 1926.

Fuller, Almon H., and R. A. Caughey, "Experimental Impact Studies on Highway Bridges," *Bull.* 75, *Iowa Engr. Exp. Sta.*, 1925.

[2] Thompson, J. T., "Static Load Tests on Pavement Slabs," *Public Roads*, Vol. 5, No. 9, p. 1, November, 1924.

under the slab. Doubtless there is some question as to the accuracy of the data available, owing to the difficulties of measuring instantaneous maximum values of the impact stresses. For the purposes of design probably it is on the safe side to assume that the impact stress produced by the maximum wheel loads carried on pneumatic tires is equal to 1.3 times that which would be produced by the static wheel load, although the dynamic load may be 1.5 of the static wheel load, or perhaps more.

TEMPERATURE STRESSES

Changes in the temperature of the concrete in a road slab cause changes in the form of the slab and may introduce stresses of considerable magnitude. For convenience these will be discussed as stresses due to linear changes and those due to warping.

Linear Changes.—Changes in the temperature of an unrestrained concrete slab produce changes in the linear dimensions, and the slab becomes longer, wider, and thicker as the temperature of the concrete rises. The changes in length and width must be taken into account in the design. If the temperature rise is great enough, the slab will expand until all of the cracks are closed, and the expansion joints, if any, will partially or wholly close. If the expansion continues after the joints and cracks have closed, the concrete will pass into a state of compression, and the stress thus produced may be sufficient to rupture the concrete, as is evidenced by those occasional failures of concrete roads or concrete bases known as "blow-ups."

A decrease in the temperature of the concrete will cause the slab to try to shorten, which will produce tensile stresses in the concrete, since the slab will meet with some resistance to sliding along the subgrade. If the stress thus produced is greater than the tensile strength of the concrete, rupture will occur. But if the slab is in sufficiently short sections, the stress will be relieved by movement of the slab.

Ordinarily the concrete road slab is not restrained against tensile stress except by the friction between the slab and the subgrade beneath it. Measurements of the magnitude of the coefficient of sliding resistance between slab and subgrade[1] indicate that it may reach a value of 2 or more and that for design purposes the value 2 should be used.

[1] GOLDBECK, A. T., "Friction Tests of Concrete on Various Subbases," *Public Roads*, Vol. 5, No. 5, p. 19, July, 1924.

The increase in the length of an unrestrained road slab of concrete due to a rise in temperature is equal to

$$L' = 12Let_r, \tag{1}$$

in which

L' = the increase in length of slab, in inches.

e = the coefficient of thermal expansion for the concrete.

t_r = the rise in temperature, in degrees Fahrenheit.

L = the length of the slab, in feet.

The maximum compressive stress that can be induced in a restrained road slab (expansion joints closed) due to a rise in temperature is

$$S_c = et_rE, \tag{2}$$

where

S_c = the compressive stress resulting from the temperature change, in pounds per square inch.

e = the coefficient of expansion of concrete, usually taken to be 0.000005 for concrete of the quality used for road slabs.

E = the modulus of elasticity for concrete, which will be taken herein as 5,000,000 for concrete of the quality used for road slabs.

t_r = the rise in temperature of the slab, in degrees Fahrenheit.

The maximum tensile stress that can be induced in a road slab of infinite length by a change of temperature is also computed by means of Formula (2). For slabs of finite length L the limit of the tensile or compressive stress is

$$S_c \text{ or } S_t = \frac{1}{2}\frac{(L)(w)(C_s)}{A_c}, \tag{3}$$

where

C_s = the coefficient of resistance between slab and subgrade ($C_s = 2$ for design purposes).

L = the length of the slab between contraction joints, in feet.

w = the weight of the slab, in pounds per foot of length.

A_c = cross-sectional area of the concrete slab, in square inches.

S_c, S_t = the tensile or compressive stress that may be produced in the concrete by a change in temperature, in pounds per square inch.

The spacing of expansion joints can be calculated from Equation (1), in which L' may be taken as the effective width of the expansion joint, usually fixed at ¾ in., L is the spacing of the joints, and t_r the increase in temperature from the time the pavement was laid to the maximum for the season in the region in which the construction is located.

If tie steel is to be provided across transverse cracks and joints, it should be of such cross-sectional area that the stress in the steel when the joint or crack is pulled open does not exceed a safe working tensile stress. That is, from Formula (3), it may be seen that

$$A_s = \frac{(w_1)(C_s)}{S_s}, \tag{4}$$

where

A_s = the area of steel required per foot of longitudinal joint, in square inches.

w_1 = the weight of a strip of pavement 1 ft. wide extending from the longitudinal joint to the nearest free edge of the pavement, in pounds.

S_s = the allowable working tensile stress in the tie steel, in pounds per square inch (usually taken at 20,000 p.s.i. for structural steel and recommended at 28,000 p.s.i. for cold-drawn wire).

The tie steel is probably subjected to shear or a combination of tension and shear by the load-transfer action when used across tongue-and-groove joints and particularly across plain joints of the dummy type. There is no possibility of computing the stresses induced in the steel by traffic loads under these circumstances. It is certain that the stress cannot be pure shear, and there is no sure method by which the active stress can be computed.

Warping.—When the upper surface of a concrete road slab is warmer than the lower, the slab becomes slightly convex transversely, and the middle portion tends to rise from the subgrade. When the upper surface is cooler than the lower, the slab becomes slightly concave transversely, and the edges tend to rise from the subgrade. It is generally accepted that the longitudinal cracks found in wide road slabs are due in part to the stress conditions set up by this warping action. When, owing to warping or other causes, the middle portion of a road slab has

been raised slightly from the subgrade, the traffic loads upon the slab could readily produce bending in the longitudinal section along the center line sufficient to rupture the ordinary road slab.

The tendency for the slab to warp longitudinally is also present; but, because of the length of the slab and the effect of tie steel at the transverse joints, the warping is restrained, with the result that eventually all concrete road slabs develop transverse cracks unless expansion and contraction joints are placed quite close together. The best information now obtainable indicates that in order to eliminate transverse cracks entirely, it would be necessary to have either expansion or contraction joints at intervals of not more than 10 to 12 ft. Whether or not this is a desirable design will be discussed in another place. Investigations of the occurrence of stresses due to warping show that they may reach very significant magnitudes, sometimes being as great as the stresses produced by loads.[1] The magnitude of the stress due to warping is dependent upon the difference in temperature between the upper and lower surfaces of the pavement slab which may in extreme cases reach 4°F. per inch of thickness, with 3°F. per inch of thickness a differential frequently reached.[2]

Stresses Due to Warping.—The stresses likely to be produced by the warping of the slab may be estimated by the following formulas, which are derived from the Westergaard analysis previously mentioned.[1]

Edge stresses:

$$\sigma_{xe} = \frac{C_x E e t}{2}. \tag{5}$$

Interior stresses:

$$\sigma_x = \frac{Eet}{2}\,\frac{C_x + \mu C_y}{1 - \mu^2}, \tag{6}$$

$$\sigma_y = \frac{Eet}{2}\,\frac{C_y + \mu C_x}{1 - \mu^2}; \tag{7}$$

[1] WESTERGAARD, HENRY M., "Analysis of Stresses in Concrete Pavements Due to Changes in Temperature," *Proc. 6th Annual Meeting*, Highway Research Board, 1926, p. 201.

[2] KELLEY, E. F., "Application of the Results of Research to the Structural Design of Concrete Pavements," *Journal of American Concrete Institute*, Vol. 10, No. 6, p. 437, June, 1939.

in which

σ_{xe} = maximum stress, in pounds per square inch, in the extreme fiber at the edge of the slab, in the direction of slab length. At the extreme edge the stress at right angles to the edge is zero.

σ_x = maximum stress, in pounds per square inch, in the extreme fiber at the interior of the slab, in the direction of slab length.

σ_y = maximum stress, in pounds per square inch, in the extreme fiber at the interior of the slab, in the direction of slab width.

E = modulus of elasticity of concrete, in pounds per square inch.

e = thermal coefficient of expansion and contraction of concrete per degree Fahrenheit.

t = difference in temperature between top and bottom of slab, in degrees Fahrenheit.

C_x and C_y are coefficients determined from the curve[1] of Fig. 85.

The Subgrade.—The portion of the roadway upon which the road slab rests is known as the *subgrade*, and of course the subgrade must support not only the road slab itself but, in addition, the traffic loads that are carried on the roadway surface. If the subgrade is very firm, it will offer high resistance to distortion of the road slab, which will have an effect upon the area of slab that will be distorted by an individual wheel load. If the subgrade is very soft, it will offer low resistance to the distortion of the road slab by wheel loads; and a relatively large area of the slab will be distorted before sufficient upward pressure is developed in the subgrade to set up a condition of equilibrium. In general, the greater the vertical distortion of the slab by a wheel load the greater the stress caused by that load, all other conditions remaining unchanged.

The extreme variation in the characteristics of subgrade soils has already been discussed at length, and it is recognized that in the absence of a definite stabilization treatment the subgrade in various sections of a road will differ considerably in stability when wet and in supporting power under any condition of saturation and that the soils in one region may differ greatly in bearing power from those in another region. It has been pointed out

[1] Derived from Westergaard's analysis by R. D. Bradbury; see "Reinforced Concrete Pavements," p. 40.

that most soils have much higher supporting power when dry than when wet. Older[1] has shown that with certain soils there is at least a certain minimum below which it is impracticable to reduce the water content by drainage. Eno[2] in a most exhaustive digest of researches in this field, supplemented by important researches in Ohio, concluded that there is as yet no basis for accurately predicting the behavior of subgrade soils, although in humid regions a neglect to observe well-known principles of

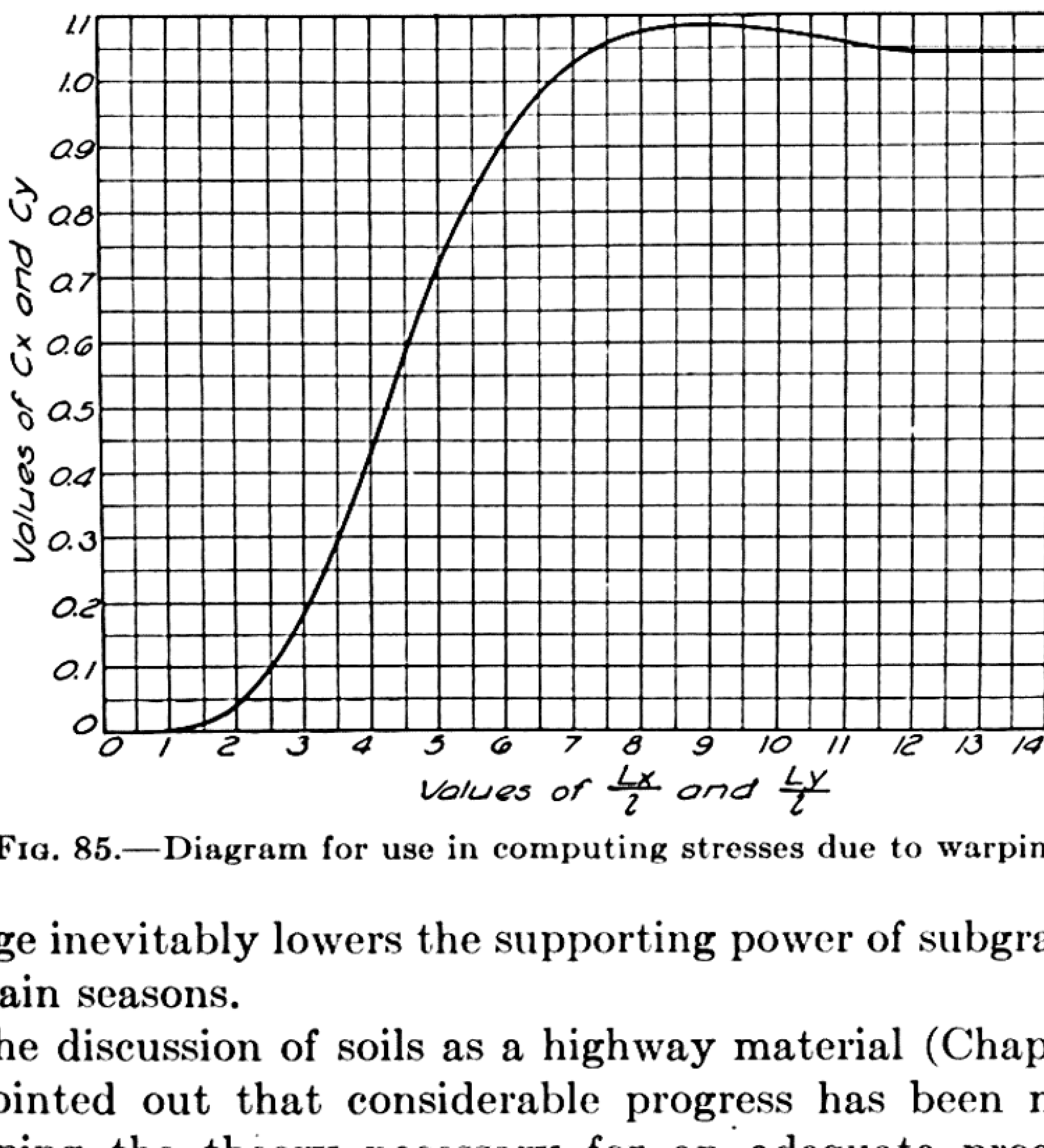


FIG. 85.—Diagram for use in computing stresses due to warping.

drainage inevitably lowers the supporting power of subgrade soils at certain seasons.

In the discussion of soils as a highway material (Chap. IV) it was pointed out that considerable progress has been made in developing the theory necessary for an adequate program of stabilization of subgrade soils. In the preparation of the subgrade for road slabs of concrete there should be a stabilization program that will insure that the soil is prepared to provide at least a certain minimum of subgrade resistance, say 10 or 15 lb. per square inch and, particularly, that an attempt be made to secure uniformity in the subgrade soil.

[1] OLDER, CLIFFORD, "Bates Experimental Road," *Bull.* 21, *Ill. Dept. Public Works*, 1924.

[2] ENO, F. H., "Highway Subsoil Investigation in Ohio," *Bull.* 39, *Univ. Ohio Engr. Exp. Sta.*, 1928.

Despite the advance in knowledge about the stabilization of soils, when one takes account of the accidents of construction, the nature of the materials, and the uncertainty as to climatic action, it is quite apparent that there is no certainty of being able to provide a uniform subgrade. In other words, truly uniform subgrade is an ideal toward which the engineer is working but which is seldom achieved in road construction.

Changes in the texture of the soil on account of water that reaches the subgrade through cracks and joints in the pavement, the effect of repeated freezing and thawing, the variation in the contact between these road slabs and the subgrade soil on account of the warping that takes place in the slab under certain temperature conditions, and the fact that even at failure the amount of distortion of a concrete road slab is very small all lead to the conclusion that in the design of road slabs of concrete, assumptions as to the amount and extent of subgrade support should always be kept on the conservative side.

In the mathematical analysis of stresses in concrete road slabs[1] discussed on page 279 there is introduced a quantity called the "radius of relative stiffness" for subgrades which is represented mathematically by the expression

$$l = \sqrt[4]{\frac{Eh^3}{12(1 - \mu^2)k}} \qquad (8)$$

in which

l = the radius of relative stiffness, in inches.
E = the modulus of elasticity for concrete (for which the value 5,000,000 is used herein).
h = the thickness of the pavement slab, in inches.
μ = Poisson's ratio for concrete, usually taken at 0.15.
k = a constant representing the load per square inch required to cause a deflection of 1 in. in the subgrade, when the total load is applied to a considerable area, such as 2 or 3 sq. ft.

For computation purposes the value of k should be selected according to the known nature of the subgrade, but for general computations use $k = 100$. Tables giving values for l for various

[1] WESTERGAARD, H. M., "Computations of Stresses in Concrete Roads," *Proc. 5th Annual Meeting*, Highway Research Board, p. 90; *Proc. 6th Annual Meeting*, p. 201.

values of the other constants are given in the report mentioned above,[1] based on $E = 3{,}000{,}000$.

Computation of Stresses Due to Load.—The Westergaard analysis provides three master formulas for use in calculating the stresses in a concrete road slab, which when $\mu = 0.15$ and $E = 5{,}000{,}000$ are as follows:[2]

$$S_i = \frac{0.3162P}{h^2}\left[4 \log_{10}\left(\frac{l}{b}\right) + 1.069\right]. \tag{9}$$

$$S_e = \frac{0.572P}{h^2}\left[4 \log_{10}\left(\frac{l}{b}\right) + 0.359\right]. \tag{10}$$

Kelley[3] found that the stress due to a corner load could be most accurately computed by the following:

$$S_c = \frac{3P}{h^2}\left[1 - \left(\frac{a\sqrt{2}}{l}\right)^{1.2}\right], \tag{11}$$

where

S_i, S_e, S_c = the maximum stress due to "interior," "edge," and "corner" loads, respectively.

h = the slab thickness, in inches.

P = the wheel load plus impact allowance, in pounds.

a = the radius of wheel load distribution, in inches.

l = the radius of relative stiffness, in inches.

b = the radius of resisting section = $\sqrt{1.6a^2 + h^2} - 0.675h$.

EFFECT OF EMBEDDED STEEL

The utility of embedded steel in concrete road and pavement slabs when of correct design and placed properly has been satisfactorily established by numerous studies[4] of the subject, and its use is almost universal. Although the benefit of steel is proved beyond question, there is a good deal of variation in the steel requirements of various specifications.

[1] *Ibid.*

[2] BRADBURY, *op. cit.*, p. 31. These formulas were also suggested by E. F. Kelley in the paper to which reference is made on page 275.

[3] *Ibid.*, p. 443.

[4] HOGENTOGLER, C. A., "Economic Value of Reinforcement in Concrete Roads," *Proc. 5th Annual Meeting*, Highway Research Board, Part II, Washington, D.C., 1925.

Purpose of Embedded Steel.—Steel is used in concrete road slabs not to add to the structural strength but to hold the faces of the slab in close contact at cracks and joints, thus aiding in the transfer of loads across the cracks and joints and preventing cracks from opening to an extent that complicates the maintenance of the pavement. If cracks develop in such a manner as to form free corners in the central portion of the slab, these are as much a source of weakness as they would be if they were at the edge of the slab. The reinforcement is presumed to provide dowels that, when the concrete slab cracks, will eliminate the possibility of unsupported corners or edges being formed either in the central portion of the slab or at the edge. This requires that the embedded steel be placed so that the bars are close enough together to accomplish that purpose. Transverse cracks are sure to develop unless contraction joints are provided at intervals of 10 or 12 ft., and possibly even then; and the longitudinal steel is intended to keep these from widening and to dowel together the two parts of the slab so that they will aid each other in carrying the loads to the supporting subgrade.

Steel Tie-bars.—The prevailing type of longitudinal joint is of the tongue-and-groove design which is effective if the faces of the slab at the joint can be kept in contact. This is accomplished by using tie-bars across the joint. The tension on these bars is that which is produced by drag on the subgrade when the slab contracts, and the magnitude can be computed from Formula (4).

If the slab is 20 ft. wide, weighs 1,900 lb. per foot of length, and C_s is taken as 2, the tension across the joint per foot of length is 1,900 lb.; and with an allowable tensile stress of 20,000 p.s.i. in the steel, the tie-bars must provide $1{,}900/20{,}000 = 0.095$ sq. in. of metal per foot of length of joint. This will require ⅝-in. round bars spaced 3.2 ft. apart ($0.307/0.095 = 3.23$) or the equivalent thereof.

If plain-faced longitudinal joints are used, the transverse bars should be close enough together to prevent excessive deflection between bars and of sufficient cross-sectional area to transmit half the load. The curvature of the slab is rather sharp near the load (see Fig. 83), and the best evidence now available indicates that the spacing of transverse tie-bars should be about 2 ft. in order to be most effective. The area of steel should be such that a single bar will transmit one-half of a wheel load, or say

5,000 lb., but there is no way to analyze the combination of bending and shear to which the tie-bar is subjected. The safe procedure is to use ⅝-in. round bars, spaced 2 ft. apart. The length of the tie-bar should be sufficient to develop bond strength equal to the tensile strength, which requires embedment of 10 diameters of the bar for deformed bars and 40 diameters for plain bars.

Longitudinal Steel.—Longitudinal steel is used primarily to provide dowels at contraction joints and at the transverse cracks that may be expected to develop in road slabs of concrete. Since there is no predicting where transverse cracks will appear, the steel is placed so as to be continuous except across expansion joints. When contraction joints are spaced properly, the longitudinal steel serves to minimize the amount of transverse cracking between joints; but the amount of steel is too small to have any considerable effect on the structural strength of the slab. Plain round bars are used for longitudinal reinforcing and are painted and greased to make sure that the concrete will not bond to them. If the concrete does bond, excessive stresses may develop in the steel at cracks, and in some cases the steel has even ruptured.

Since the wheel loads are most frequently applied near the edges of the traffic lanes, it is customary to place the longitudinal bars from 6 in. to 1 ft. from each edge of each traffic lane. Sometimes a third bar is placed along the middle of the lane. In most designs, ¾-in. smooth round bars are used. Deformed bars are not used for longitudinal reinforcing.

If the reinforcement is in the form of a woven or welded fabric, the size and spacing of the members are adjusted to provide ample tie steel across joints and cracks that may occur. The composition of the fabric is determined by calculating the area of steel required to carry the tensile stress estimated to be probable. For example, if the expansion joints are 50 ft. apart, the tension (page 273) midway between the expansion joints of a 7½-in. slab will be 4,887 lb. per foot of width of slab. With a working tensile stress of 28,000 p.s.i., a fabric providing 0.142 sq. in. in cross-section per foot of width will be required.

Value of Dowel Steel.—Investigations of the deflection of concrete road slabs indicate that when adjacent slabs are connected by steel dowel bars, the capacity of the doweled joint to transmit shear varies greatly and that, in consequence, the deflec-

tion on the load side of a crack may be greater than on the unloaded side of the crack. In some cases investigated, not more than 5 per cent of the load was transmitted across the crack by shear and friction, and in other instances as much as 40 per cent was transmitted.[1] This variation is doubtless due to the extent to which the crack opens when the slab shortens from the effect of lowered temperature and shrinkage. The load transmitted across the doweled cracks and joints when the design is correct and the construction well done is believed to be in excess of 25 per cent of the total.

A further consideration in connection with dowel steel is that Westergaard's analysis (page 279) indicates that the critical stress in a road slab of concrete is developed when the load is at the margin of the slab where there is no adjacent slab to aid in carrying load or at a crack without dowels that is wide enough to permit the slab to deflect freely. Dowel steel will eliminate this particular stress condition except at the margin of the slab, and there it is taken care of by the thickened section.

THE DESIGN OF ROAD SLABS

The design of road slabs of concrete involves consideration of the several factors that have been discussed in previous sections and perhaps others. Some of the problems presented in this connection have been under study for 20 years or more, and in recent years many investigators have been working on problems related to concrete road design. Nevertheless there is not as yet a complete rational theory for the design of concrete road slabs. However, by exercising reasonable judgment and weighing carefully the results of published researches, adequate designs can be prepared for concrete road slabs for any forecasted traffic load.

The Corner Load Theory.—The intersection of the transverse joints or cracks with the edge of the concrete road slab results in the formation of corners as indicated at *A* in Fig. 84. If the transverse crack or joint is wide enough, the loaded corner will deflect without affecting the adjacent slab, behaving as a special form of cantilever beam. Goldbeck[2] pointed out that on this

[1] "The Structural Design of Concrete Pavements," *Public Roads*, Vol. 17, No. 7, p. 143, September, 1938.

[2] GOLDBECK, A. T., "Thickness of Concrete Slabs," *Public Roads*, Vol. 1, No. 120, p. 34, April, 1919.

assumption a formula could be derived for computing the thickness of the slab to carry an assumed load. The derivation follows.

It is assumed that the slab is of uniform thickness and that rupture will occur along a vertical plane at an angle of 45 deg. with the edge of the pavement, at a distance x from the corner, as shown at A in Fig. 84, and that the corner carrying the load is free to deflect independently of the portion of the slab beyond the transverse crack or joint (no embedded steel and no friction between the two portions of the slab).

The bending moment of the load is

$$M_1 = Wx. \tag{12}$$

The resisting moment of the plain concrete of uniform thickness, without exceeding the selected working stress, is

$$M_r = \frac{S_c I}{c} = \frac{S_c}{c}\frac{h^3 x}{6} = \frac{1}{3}S_c h^2 x, \tag{13}$$

where

W = one equivalent static wheel load, in pounds.

S_c = the safe working flexural modulus of the concrete, in pounds per square inch.

I = moment of inertia of the plane of rupture.

h = thickness of the slab, in inches.

$c = h/2$.

Then

$$Wx = \frac{S_c h^2 x}{3};$$

$$h = \sqrt{\frac{3W}{S_c}}. \tag{14}$$

In the analysis of the data from the Bates road tests,[1] Older made use of Formula (12), and subsequently Frank T. Sheets adapted this formula[2] to a method of design of concrete road slabs.

The corner formula is derived on the assumption that the subgrade support (page 276) of the slab should be neglected and does

[1] Older, Clifford, "Highway Research in Illinois," *Trans. A.S.C.E.*, Vol. 87, p. 1206, 1924.

[2] Sheets, Frank T., "Concrete Road Design," *Bull.* 2, *Highway Planning and Design Ser.*, Portland Cement Association, Chicago, Ill.

not take account of the effect on stresses of the load being applied over an appreciable area, as is the case with loads on pneumatic tires. The corner load formula must be looked upon as providing against the maximum possible stress from a given load.[1]

Since "corners" occur frequently along the edge of a pavement slab and may develop anywhere in the slab from the advent of transverse cracks, the corner load formula is in fact an edge load formula for a slab of uniform thickness. The load formula for the thickened-edge type requires certain special considerations.

The corner load formula assumes that the loaded corner is free to deflect without causing deflection of the adjacent corner of the pair that will always exist at a transverse crack or joint. If the slab is constructed with embedded steel (page 281) of the correct design, one corner of a pair cannot deflect without the other also deflecting. The extent of the deflection of the unloaded corner will depend upon the efficiency of the method of load transfer provided in the construction.[2] From all the data so far available it seems reasonable to assume that the unloaded corner of a pair can be expected to carry 25 per cent of the load by transfer across the joint from the loaded corner. The corresponding reduction in load on the loaded corner will modify Formula (14), because only 75 per cent of W must be carried on the loaded corner. Therefore

$$h_c = \sqrt{\frac{2.25W}{S_c}} \qquad (15)$$

may be used as a working formula in the development of tentative designs for the edge thickness of road slabs, and this thickness must extend in at least 30 in. from the edge, or equivalent load-carrying strength be provided by some type of thickened-edge design.

In the area between the margins of the slab, corners exist at the junction of transverse cracks and joints with the longitudinal

[1] Spangler, M. G., "Stresses in Concrete Pavement Slabs," *Proc. 15th Annual Meeting*, Highway Research Board, Washington, D.C., 1935, p. 122.

Spangler, M. G., and F. E. Lightburn, "Stresses in Concrete Pavements," *17th Annual Meeting*, Highway Research Board, Washington, D.C., 1937, p. 215.

Teller, L. W., and E. C. Sutherland, "The Structural Design of Concrete Pavements," *Public Roads*, Vol. 17, No. 7, p. 143, September, 1936.

[2] Teller and Sutherland, *op. cit.*, p. 175.

joints and by the intersection of incidental cracks. The corners thus formed differ from corners that appear at the edge of the slab in that the adjacent slab abuts the two sides of the wedge forming the corner. If the slab is provided with embedded steel of suitable design and distribution, these adjacent slabs may be counted upon to carry 50 per cent of the load that rests on the corner. The stress due to the load on the slab may be thought of as a special case of the corner load already discussed, and therefore

$$h_i = \sqrt{\frac{1.5W}{S_i}}. \tag{16}$$

It is convenient at times to use the ratio of interior thickness to edge thickness in preparing tentative designs for slabs. The ratio is $\sqrt{1.5/2.25} = 0.8165$.

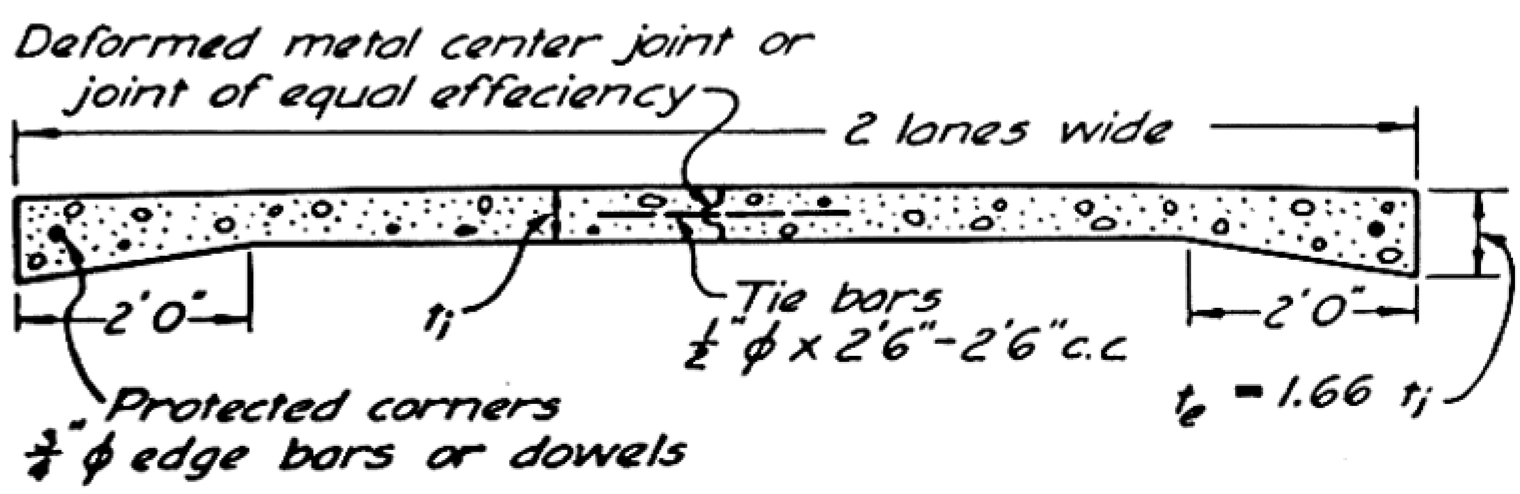


FIG. 86.—Illustrating a road slab designed for balanced stresses.

If the joint efficiency were taken at 40 instead of 50 per cent, the foregoing ratio would be 0.866. Since the ratio is used for preparing tentative designs, and there is some experience indicating a tendency toward too thin slabs for the heavy-duty roads, it seems wise to use the ratio 0.85 as a compromise between 0.8165 and 0.866.

Thickened-edge Slabs.—The thickened-edge type of cross-section has been widely used for many years as a result of the character of the failures observed on the Bates road tests.

The Arlington tests by the Bureau of Public Roads[1] indicated that a section to be well balanced against load stresses should have the form shown in Fig. 86. Formula (14) is valid when the slab is provided with sufficient embedded steel to insure at least the amount of load transfer across the transverse and longitudinal joints assumed in its development. If it is used for determining the thickness of the interior portion of the slab,

[1] *Public Roads*, December, 1935.

the extreme edge thickness (Fig. 86) can be fixed by multiplying the computed thickness for the interior by a suitable factor. On the basis of Arlington test results, that factor would be 1.66. But for areas of considerable precipitation and cycles of alternate freezing and thawing, with the resulting uncertainties as to the subgrade support, a lower ratio is believed to be in order. There is a considerable mileage of 10–7–10 pavement (ratio $h_e/h_i = 1.43$) in which there is some evidence that the edge is stronger than the interior portion. For preliminary designs it is suggested that the ratio h_e/h_i be taken at not more than 1.5 when the thickened edge is of the kind shown in Fig. 86.

Applications.—The application of these several ideas and empirical formulas will be illustrated by some computations that may serve to emphasize the principles involved. Design procedure in engineering offices may be made considerably simpler than as here illustrated by using tables and diagrams for checking stresses, but all short cuts will be avoided herein.

Problems

1. Design a two-lane concrete road slab (lanes 10 ft. wide) for a location in which the maximum static wheel load that recurs with sufficient frequency to require consideration is 8,000 lb., carried on high-pressure pneumatic tires. It is expected that concrete will be provided for which a safe working flexural modulus of rupture is 350 p.s.i. The subgrade soil will be treated or manipulated in such a way as to provide at the time of construction a value of k equal to 100. The concrete slab may be of the thickened-edge type or of uniform thickness according to the relative cost. Expansion joints are to be placed at intervals of 45 ft., with contraction joints spaced 15 ft. apart. The slab will be provided with embedded steel in correct amount.

Static wheel load 8,000 lb. Impact factor 1.3.

Equivalent static load (8,000) (1.3) = 10,400 lb.

Step 1.—Compute the thickness of slab required for a corner wheel load of 8,000 lb. by use of Formula (15).

$$h_c = \sqrt{\frac{(2.25)(10{,}400)}{350}} = 8.18 \text{ in.} \quad \text{Use 8 in. for trial design.}$$

Step 2.—Approximate the thickness required for the interior portion (the part more than 2½ ft. from joints or edges) by multiplying the corner thickness computed in Step 1 by 0.85 (page 285)

$$h_i = (8)(0.85) = 6.80 \text{ in.}$$

Step 3.—Estimate the maximum stress in the interior portion due to the wheel load, from Formula (9). For this purpose $k = 100$; and $l = 34.0$, $b = 7.60$, $a = 8$ in.; and $h_i = 6.8$ in., $P = 8{,}000 \times 1.3 = 10{,}400$ lb.

$$S_i = (0.3162)\frac{10{,}400}{(6.8)^2}\left[4 \log \frac{34.03}{7.60} + 1.069\right],$$
$$S_i = 269 \text{ p.s.i.}$$

Step 4.—Estimate the stresses in the interior portion of the slab due to warping, using Formulas (6) and (7).

$$t = 3°\text{F. per inch thickness,} \quad l = 34.0, \quad L_x = 180, \quad L_y = 120,$$
$$C_x \text{ (Fig. 85)} = 0.8, \quad C_y \text{ (Fig. 85)} = 0.3.$$
$$\sigma_x = \frac{(5{,}000{,}000)(0.000005)(20)}{2}\left(\frac{0.8 + (0.15)(0.3)}{1 - 0.0225}\right).$$
$$\sigma_x = (250)(0.762) = 216 \text{ p.s.i.}$$
$$\sigma_y = (250)(0.430) = 107 \text{ p.s.i.}$$

Step 5.—Determine the approximate total stress under maximum stress conditions in the interior portion of the slab by adding the load stresses and the stress due to warping.

$$S_i = 269 \text{ p.s.i.}$$
$$\sigma_x = 216 \text{ p.s.i.}$$
$$\sigma_y = 107 \text{ p.s.i.}$$
Total stress transversely, $269 + 107 = 376$ p.s.i.
Total stress longitudinally, $269 + 216 = 485$ p.s.i.

Step 6.—Determine the edge thickness to use if the cross-section is to be of the type shown in Fig. 86. This is a matter of judgment based on experience, as has previously been noted. For this illustration the edge thickness will be provided by using the ratio of edge thickness to interior thickness of 1.5. Hence the edge thickness of this slab will be (1.5) (6.8) = 10.2 in.

The foregoing covers the general conditions that result in the use of the 10–7–10 cross-section. In some areas this section has suffered damage at the intersection of transverse cracks and the longitudinal joint where corners have failed in considerable numbers. Here the weakness may be in the provisions for tie steel across the longitudinal joint; poor handling of the concrete along the longitudinal joint, resulting in weak places; or loads considerably in excess of those contemplated by the design.

Step 7.—Investigate the thickness required if the slab is to be constructed to a cross-section of uniform thickness. The critical stresses are at the corners along the margins of the slab. The computation in Step 1 gives the thickness required at the edge and consequently the thickness of the slab for a design of cross-section of uniform thickness. Whether to use 8 in. or the next standard form width, 8.5 in., or to adjust the form for a thickness of 8.17 in. as computed will be determined by the results of check computations.

Step 8.—Check the stresses in the concrete due to a load on a corner, using Formula (11), remembering that it is assumed that 25 per cent of the load will be carried by the corner adjacent to the loaded one. As in Step 3, $a = 8$ in., $k = 100$, $P = 10{,}400$, and h will be taken at 8 in.

$$S_c = \frac{(0.75)(3)(10{,}400)}{64}\left[1 - \left(\frac{(8)(1.414)}{38.4}\right)^{1.2}\right],$$
$$S_c = (366)(0.7693) = 282 \text{ p.s.i.}$$

Step 9.—Check the load stress at the unbroken edge, using Formula (10) in which $P = 10{,}400$, $h = 8$ in., $a = 8$ in., $l = 38.4$, $(k = 100)$, $b = 7.5$.

$$S_e = 0.572\frac{10{,}400}{64}\left[4 \log \frac{38.4}{7.5} + 0.359\right],$$
$$S_e = (0.572)(162)[(4)(0.7093) + 0.359],$$
$$S_e = 297 \text{ p.s.i.}$$

Step 10.—Check the load stress in the interior of the slab, using Formula (9), in which $P = 10{,}400$, $h = 8$ in., $a = 8$ in., $l = 38.4$, and $b = 7.5$

$$S_i = 0.31625\frac{10{,}400}{64}\left[4 \log \frac{38.4}{7.5} + 1.0693\right],$$
$$S_i = (0.31625)(162)(3.9) = 199.6,$$
$$S_i = 199 \text{ p.s.i.}$$

Step 11.—Check the section for warping stresses. It is known that these are negligible at the corners, and the unit stresses due to load are so low in the interior portion of the slab that there is little probability of the warping stresses adding enough to be of consequence. But at the edges the warping stresses may be significant: $L_x = 180$, $L_y = 120$, $l = 38.4$, $C_x = 0.62$.

$$\sigma_{xe} = \frac{(5{,}000{,}000)(0.000005)(24)(0.6)}{2},$$
$$\sigma_{xe} = (300)(0.6) = 186 \text{ p.s.i.}$$

The maximum probable stress at the edge of the slab due to loads and warping is 483 p.s.i. This may be considered too high and result in an objectionable number of transverse cracks as time passes. If the thickness of the section is increased to 8.5 in., the combined stress will be $S_e = 270$, $\sigma_e = 185$, total 455 p.s.i. The maximum temperature stress does not recur frequently, and only when the maximum temperature stress and the maximum load stress came simultaneously would the total stress reach the total computed above. The design is probably safe and reasonable if these infrequent high stresses do not exceed about 75 per cent of the modulus of rupture of the concrete.

CONCRETE FOUNDATIONS FOR WEARING SURFACES

Road slabs of concrete are used extensively as the foundation for a wearing surface of blocks or an asphaltic mixture, and the effect of any of these surfaces upon the load-carrying capacity of the concrete base is a matter of considerable significance.

Effect of Wearing Surfaces on Structural Strength.—The wearing surface can affect the load-carrying capacity of the concrete base by adding to its structural strength and by distributing the wheel load to the base over an area greater than the area of contact between tire and wearing surface.

Cement-grout filled block surfaces undoubtedly do add to the structural strength of the base, and this may most conveniently

be taken into account by considering the wearing surface to be equivalent to a certain thickness of concrete slab required to carry the wheel loads without aid from the wearing course, and the thickness thus obtained is then reduced by an amount that experience indicates to be justified by the effect of the wearing surface on the structural strength of the combined structure. A rule that seems fairly safe is to reduce the computed thickness of the concrete slab by an amount equal to one-half the thickness of the block surface, exclusive of the bedding course.

Block surfaces filled with bituminous materials and the various hot-mixed bituminous surfaces do not appear to add to the structural strength of the section sufficiently to justify considering them as affecting the design of the base slab. Probably these surfaces do add somewhat to the ability of the base slab to carry moving loads, but it is impossible to estimate what the amount may be.

Load Distribution by Wearing Surfaces.—The area of application of the load to the slab is taken into account in the design of road slabs (page 269) and is a significant factor. A wearing surface of any kind tends to distribute the load to a greater area of slab than that actual contact area between tire and wearing surface which has considerable bearing on the design of foundation slabs. The various kinds of block surfaces are more effective in accomplishing this than the thinner and more plastic bituminous surfaces. It is probably true that block surfaces tend to carry loads across cracks in the base in somewhat the same way as dowel steel, which may account for the stability of some of the block pavements with rather thin concrete slabs under them. There is little evidence to indicate that wearing surfaces are effective in minimizing the impact reactions on the base course.[1]

All the dependable evidence now available tends to point to the necessity of designing concrete slab foundation courses on about the same basis of load distribution as slabs to be used as wearing surfaces. Undoubtedly such a slab has a somewhat higher factor of safety than that which is introduced into the design, but the effects of the wearing surface on load distribution are so little understood that they had better be neglected in the design.

[1] Buchanan and Reid, *loc. cit.*
Fuller and Caughey, *loc. cit.*

Influence of Wearing Surface on Temperature Effects.—It seems to be clear that a wearing course of appreciable thickness laid over a concrete slab tends to insulate the concrete from those differences in the temperature of the upper and lower portions of the slab which are the cause of warping. But the concrete slab will be subjected to changes in temperature with the seasons and consequent linear changes in dimensions to about the same extent as slabs not covered with wearing courses. Contraction cracks will form and in many cases will extend through the wearing course. Buckling due to expansion is not uncommon and usually carries with it the rupture of the wearing course.

Design of Pavement Foundations of Concrete.—The foregoing seems to indicate that the correct procedure in the design of concrete pavement foundations is about as follows:

1. The concrete base for rural highways to be surfaced with bituminous wearing courses or bituminous-filled block surfaces should be designed exactly as though they were to be used as wearing surfaces, except that (*a*) the expansion joints should be of the air-gap type instead of mastic-filled, (*b*) contraction joints can be omitted, (*c*) the longitudinal center joint should be omitted.
2. The concrete base for conventional full-width pavements on city streets can be designed on the assumption that there will be no edge loads; otherwise the design follows the rules set forth in 1 above.
3. The concrete base for block pavements that are to be filled with cement grout can be designed as though no wearing surface were to be added and then be reduced by one-half the thickness of the wearing surface exclusive of bedding course.

CHAPTER XII

PORTLAND-CEMENT CONCRETE ROADS AND PAVEMENTS

Portland-cement concrete was laid as the wearing surface for streets in Grenoble, France, in 1876; in Bellefontaine, Ohio, in 1894; and in LeMars, Iowa, in 1904; and these pavements were still in service in 1930. It was not until about 1910, however, that this type of pavement began to attract widespread attention among engineers and public officials in the United States. Since that time the concrete pavement has increased in importance at a phenomenal rate, and an immense amount of research has been undertaken with a view to standardizing its construction. Although a few installations have been made in which relatively small precast blocks or slabs of concrete were used, the more common method has been to construct the pavement in place in slabs of considerable area.

Cross-sections.—Applications of the principle of design that were discussed in Chap. XI lead to a few normal cross-sections adapted to meet the needs of the traffic and the drainage conditions that are encountered on average paving projects. These typical concrete pavement cross-sections are shown in Fig. 87.

Type 1.—The most widely used cross-section for rural highways is the thickened-edge type without marginal curbs, illustrated in Fig. 87. The dimensions of this type of pavement will depend upon the traffic and the foundation soil as was discussed in Chap. XI, but for trunk-line highways serving traffic from metropolitan areas the most usual design is with an edge thickness of 10 in. and an interior thickness of 6.7 in.; note, however, that a good many highway departments increase the thickness of the interior portion of the slab to 7 or 7.5 in., thus unbalancing the design in favor of the interior portion of the slab. This is done because experience seems to indicate that corner breaks are more prevalent in the interior portion of the slab than they are along the edges, and consequently there is an arbitrary thickening of the paving in the interior portion to take care of the

factor of uncertainty in the design. The edge thickness is gradually reduced to the thickness of the interior portion in a distance varying from a minimum of 2 to a maximum of 4 ft. according to the ideas of the responsible designer.

For trunk highways in areas not subjected to metropolitan traffic and not constituting part of cross-county trunk routes,

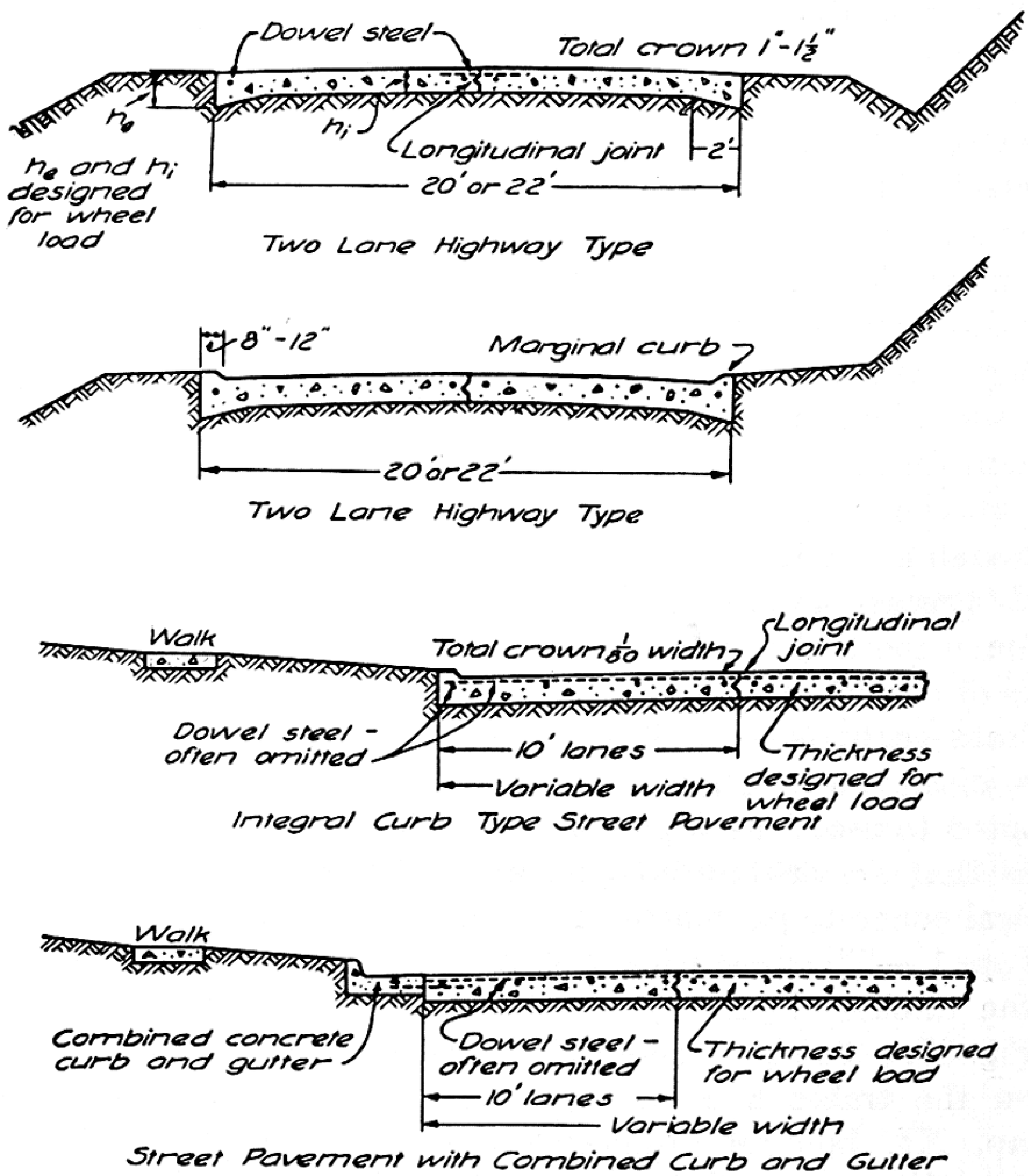


Fig. 87.—Typical cross-sections for concrete roads and pavements.

the dimensions of the slab may be somewhat less than for roads in metropolitan areas. In any case, however, 20 ft. is the minimum width that should be constructed, and a 9-in. edge is probably the minimum. This would call for a thickness of 6 in. in the interior portion of the slab. On county trunk highways the width most widely used is 18 ft., and the cross-section is 7–5.7–7 or about that.

Type 2.—In certain locations it is desirable to carry the storm water on the pavement slab to drainage outlets constructed for the purpose and so arranged as to prevent erosion by eliminating the possibility of storm water crossing the shoulder or flowing down the slide slope of embankments. When a pavement is constructed on a grade, it is preferable to carry the storm water in the gutter at the edge of the pavement slab to the bottom of the hill or to the beginning of the fill section and then dispose of it through a flume or storm-water inlet instead of utilizing the conventional side ditch. Likewise, sections of pavement on embankments may be constructed with a marginal curb, and the water carried to flumes that will let it down to the ditch level without erosion. Wherever this problem is acute the cross-section of the style shown in Fig. 87 is widely used. It should be noted that when the marginal curb is employed the thickness of the concrete slab at the edge of the pavement is not decreased but that, on the contrary, the curb is simply an addition to the edge of the standard slab.

The marginal-curb section is used for rural highways wherever topographical conditions make it advisable to carry drainage water on the pavement slab.

Type 3.—The street-pavement section with integral curbs is widely used in municipal practice; and since there is little traffic near the curb, and the curb is of sufficient size to add materially to the stability of the edge of the pavement, the cross-section of uniform thickness is generally employed for city streets.

Type 4.—The concrete pavement with a combined concrete curb and gutter is sometimes used in municipal work instead of the integral-curb type. To give the necessary stability at the edge of the pavement slab, it should either be designed with the thickened edge or else doweled to the gutter slab.

Expansion Joints.—An expansion joint is a gap in the pavement slab filled with some plastic substance that will squeeze out when the ends of the slab creep toward each other under the influence of the expansion from heat and moisture. The simplest type of expansion joint is formed with a sheet of bituminous plastic set in place before the concrete is deposited and, after the concrete has been finished, trimmed to the surface of the finished roadway. The width of these joints ranges all the way from ½ to 2 in., and in some cases expansion joints have been built in experimental pavements of a width of 3 or 4 in. From the standpoint

of the maintenance of the joints and the riding quality of the pavement, it is desirable to limit the width of the expansion joints to about 1 in. The expansion-joint fillers have the annoying property of squeezing out of the joint as the ends of the pavement approach each other in hot weather, and the filler that has exuded on the surface forms a bump which is quite annoying to traffic.

In recent years there have come on the market a number of expansion joints consisting of dowel bars and a space for the filler in a complete assembly which is ready for installation in the pavement. There are a number of types of these joints, all having similar characteristics although differing in the details of design, and none of them has proved to be particularly superior to the joints made with the premolded mastic filler. Their use has been increasing, however, in recent years because of the convenience of installation. The cost is considerably greater than that of the conventional expansion joint.

The most common type of filler for the expansion joint is the premolded sheet of bituminous mastic which is furnished in a variety of thicknesses and widths. The bituminous mastic is set in place on the subgrade before the concrete is placed and is held in position by means of an installing device so designed that the finishing machine can operate over the top of the filler. After the finishing operations have been completed, the installing device is removed, and the concrete over the joint filler and along the sheet of mastic is finished with an edging tool.

The old-style expansion joint, consisting of a form placed on the subgrade to be withdrawn after the concrete finishing operations have been completed and the space filled by pouring in asphalt, is obsolete except for very special conditions.

Contraction Joints.—It was pointed out in the discussion of stresses in road slabs of concrete that transverse cracks may often be attributed to stresses induced by warping, or by conditions produced by warping, that are favorable to the cracking of the pavement under traffic loads. The best information obtainable up to the present time indicates that stresses of dangerous magnitude result from warping when the slab is restrained in the longitudinal direction, as is the case between expansion joints when the individual sections of the slab are more than 10 or 12 ft. long. It is coming to be the practice to relieve the stresses due to curling as well as to provide for the relief of stresses due

to the contraction of the pavement by providing contraction joints at rather frequent intervals. The contraction joint is in reality a plane of weakness in the concrete which insures that under curling or contraction the crack that occurs will be regular and readily maintained against raveling. The plane of weakness in the concrete may be formed in one or the other of two ways. The first method is to set on the subgrade a metal plate extending within 1½ or 2 in. of the surface of the concrete and covering it with the concrete. When the pavement slab contracts, the comparatively weak section directly over the plate will rupture, and the contraction crack will be a straight line across the pavement at the place determined upon.

In order to provide for more ready maintenance of this type of crack the so-called "dummy" joint is frequently used instead of the separator plate. The dummy joint is formed by cutting into the surface of the concrete after the finishing has been completed by means of a wedge-shaped plate which is driven down into the concrete a depth of 2½ or 3 in. and then withdrawn. The crack thus formed is neatly finished with an edging tool and may be filled with bituminous material to aid in avoiding spalling at the surface. The contraction crack will generally open at the plane of weakness provided by the dummy joint, but curling stresses may produce cracks between dummy joints if they are spaced too far apart. In some cases the dummy joint is protected by inserting into it a preformed sheet of bituminous mastic similar in composition to the expansion joint filler but of the proper thickness and shape to be placed into the dummy joint.

Expansion and Contraction Joints in City Pavements.—Whatever may be the theoretical considerations involved in the spacing of expansion joints on rural highways, the expansion-contraction problem presented in the design of the pavement for streets is fairly definite, since manholes, valve boxes, lamp-holes, and many other types of street structures extend from the underground services up through the pavement slab. It is highly objectionable if there is any considerable movement of the slab itself, as such movement will inevitably disturb the structures that extend up through the pavement. Moreover, at street intersections and frequently at alley intersections the curb alongside the pavement as well as the pavement itself has an apron which extends back into the intersecting street or alley. Any movement of one pavement will inevitably disturb the

intersecting one. Expansion joints in street pavements should therefore be placed in four directions at each intersection so that any movement of the slab on one street will not disturb the pavement on the intersecting street. If the distance between intersecting pavements such as street or alley returns is in excess of about 60 ft., there should be an intermediate expansion joint. As a matter of fact, the best practice seems to be to place transverse expansion joints in municipal pavements at intervals not greater than 35 or 40 ft.

It is imperative that the expansion joints in the curb, whether it is separate curb and gutter or integral curb, be continuous with the expansion joints in the pavement itself. If the pavement is free to move independently of movement in the curb, there are likely to be disturbances where the pavement slab abuts the gutter and catch-basin, or other drainage structures that are partly in the gutter and partly in the pavement slab itself will be displaced by the movement.

The expansion joint arrangement for large intersections sometimes becomes rather intricate, but it is wise to provide ample relief for expansion in such areas. However, care should be exercised to avoid having expansion joints intersect at angles less than approximately 90 deg.

Contraction joints in municipal pavement would be placed according to the rule for contraction joints in rural highway pavements.

Longitudinal Joints.—In order to minimize stresses due to transverse curling of pavement slabs, longitudinal joints are constructed in the concrete slab so as to divide it into strips not more than 10 or 12 ft. wide. The usual practice is to employ the longitudinal joint at the edge of each traffic lane, and the width of the traffic lane in present-day design ranges from 10 to 12 ft. If the pavement is more than four lanes wide, the longitudinal joint at the middle should be an expansion joint with the edge of the pavement thickened.

The preferred type of longitudinal joint consists of a tongue-and-groove type of construction with transverse deformed reinforcing bars extending across the joint so as to hold the two parts of the slab in close contact at the longitudinal joint. The tongue and groove is formed by a metal sheet which is set on the subgrade before the concrete is placed. This plate does not extend entirely through the concrete but, on the contrary, is

low enough to permit the finishing machine to work over it.

Tests made to determine the extent to which the load is transmitted across joints of this type indicate that under the most favorable conditions it is not to be expected that more than 35 or 40 per cent of the wheel load on one side of a joint of this type will be transmitted to the other side, although theoretically it might be expected that the adjacent slab would carry half of the wheel load. Poorly constructed longitudinal joints will carry no more than 10 or 15 per cent of the load applied on the adjacent slab.

There are designs in which the longitudinal joint is formed by means of a device on the finishing machine that forms the dummy joint already described and in which a filler is placed subsequently. Here, again, transverse steel is provided to make sure that when the longitudinal crack actually occurs, which will be within a few months after the pavement has been placed in service, the two halves of the slab will be held together. Experience seems to indicate that, in spite of the ragged nature of the fracture that occurs under these circumstances, a joint of this type is not very efficient in transmitting deflection.

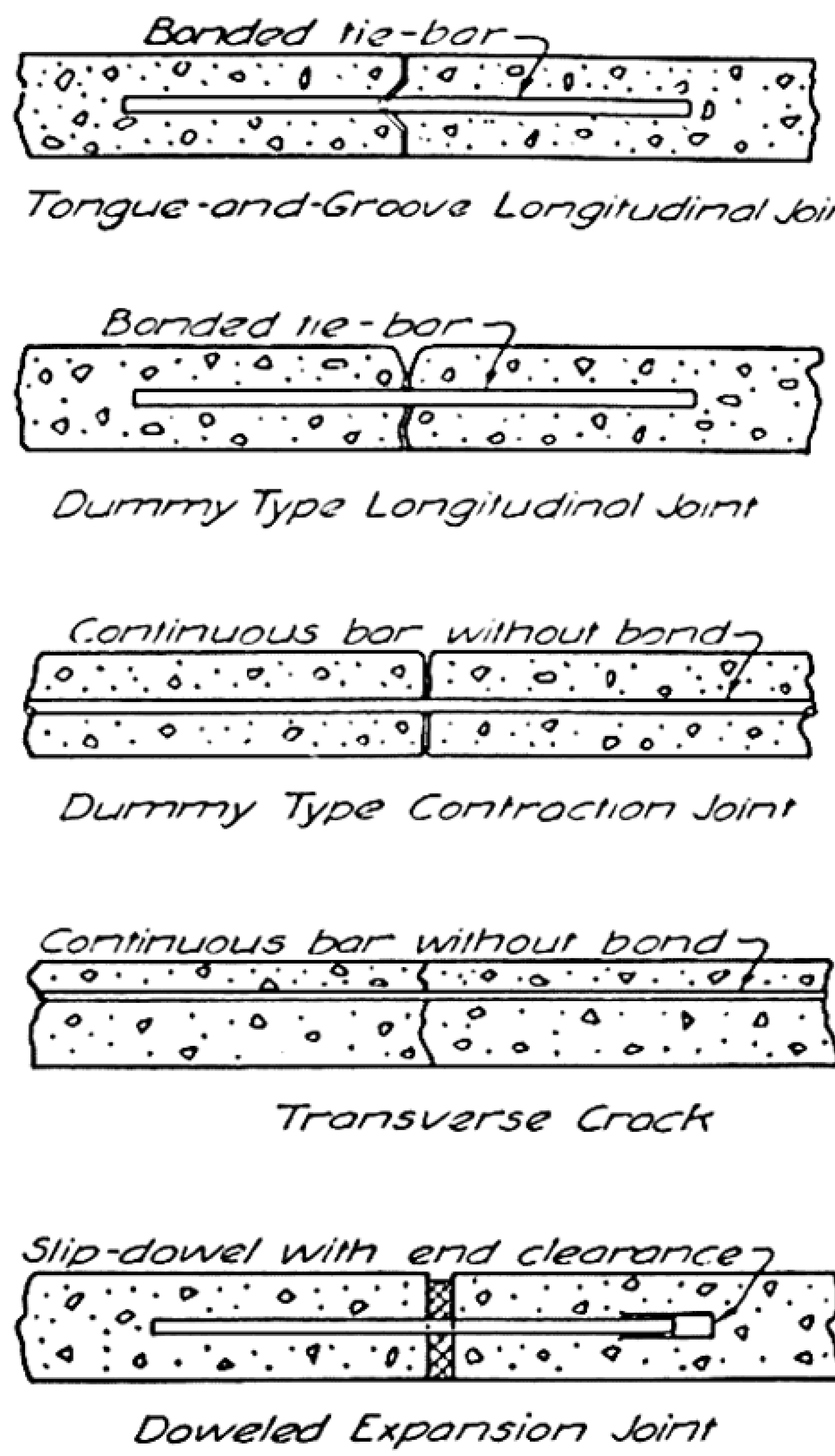


Fig. 88.—Some standard forms of joints for concrete roads.

With all types of longitudinal joints it is imperative that dowel bars be employed to hold the slab tightly together at the longitudinal joint. For this purpose deformed bars ½ or ¾ in. in diameter and 4 ft. long are employed, and they are spaced at intervals of 2 to 4 ft., depending upon the ideas of the designer as to the efficiency that he can secure.

Embedded Steel.—Embedded steel in concrete road slabs serves two functions: (1) the function of a dowel bar to insure

that when a crack appears in the concrete the dowel bar will be there to transmit the load from one side of the crack to the other and likewise that where contraction cracks are provided for, there be dowel steel across these cracks to transmit loads from one side to the other. This is the function of the longitudinal bar that is placed in the concrete road slab. These bars are placed near the outer margin of the slab. Longitudinal bars are sometimes placed along the longitudinal joint, but the feeling of most engineers seems to be that the joint-forming plate used with the tongue-and-groove type of longitudinal joint serves as the dowel bar for transverse cracks.

Since the longitudinal bar is intended to serve the purposes of a dowel, it is imperative that precautions be taken to prevent its bonding with the concrete. This is accomplished by painting and greasing the bars. The method of estimating the size of longitudinal bars to employ has already been discussed.

(2) The other type of embedded steel is the short bar used to insure that the slab on the two sides of a longitudinal joint is held close together—in other words, that the longitudinal joint is prevented from widening. For this purpose it is desirable to employ a bar that will have the maximum bond in the concrete, and generally the deformed type of reinforcing bar is employed for this purpose. There is some evidence, however, that the ordinary smooth bar will acquire sufficient bond for the purpose.

An entirely different theory of embedded steel is held by some engineers who believe that they will obtain the best results by employing steel reinforcing mats or mesh, made up of bars of smaller sections than the dowel bars heretofore discussed and with the reinforcing materials distributed throughout the slab. For this purpose mats of mesh reinforcement are available in various weights so that the engineer has some choice of quantity of reinforcing to insert in his pavement slab. The bar-mat type of reinforcing has perhaps been more widely used in municipal pavements than it has on rural highways, but it has been employed extensively for all types of highway construction.

The dowel bars used at expansion joints are smooth bars about 30 in. long, one end being bonded in the concrete at the end of a slab so as to extend across the expansion joint through holes punched in the mastic filler. The other end of the bar extends into a snug-fitting metal sleeve which is bonded in the concrete at the end of the adjacent slab. The principle is that the bar

must be able to slide in and out of the sleeve as the slabs expand or contract. The whole purpose of the sliding dowel bar is to transmit loads across the expansion joint. These bars are really effective only when the bars are about ¾ in. in diameter and spaced about 1 ft. apart along the joint.

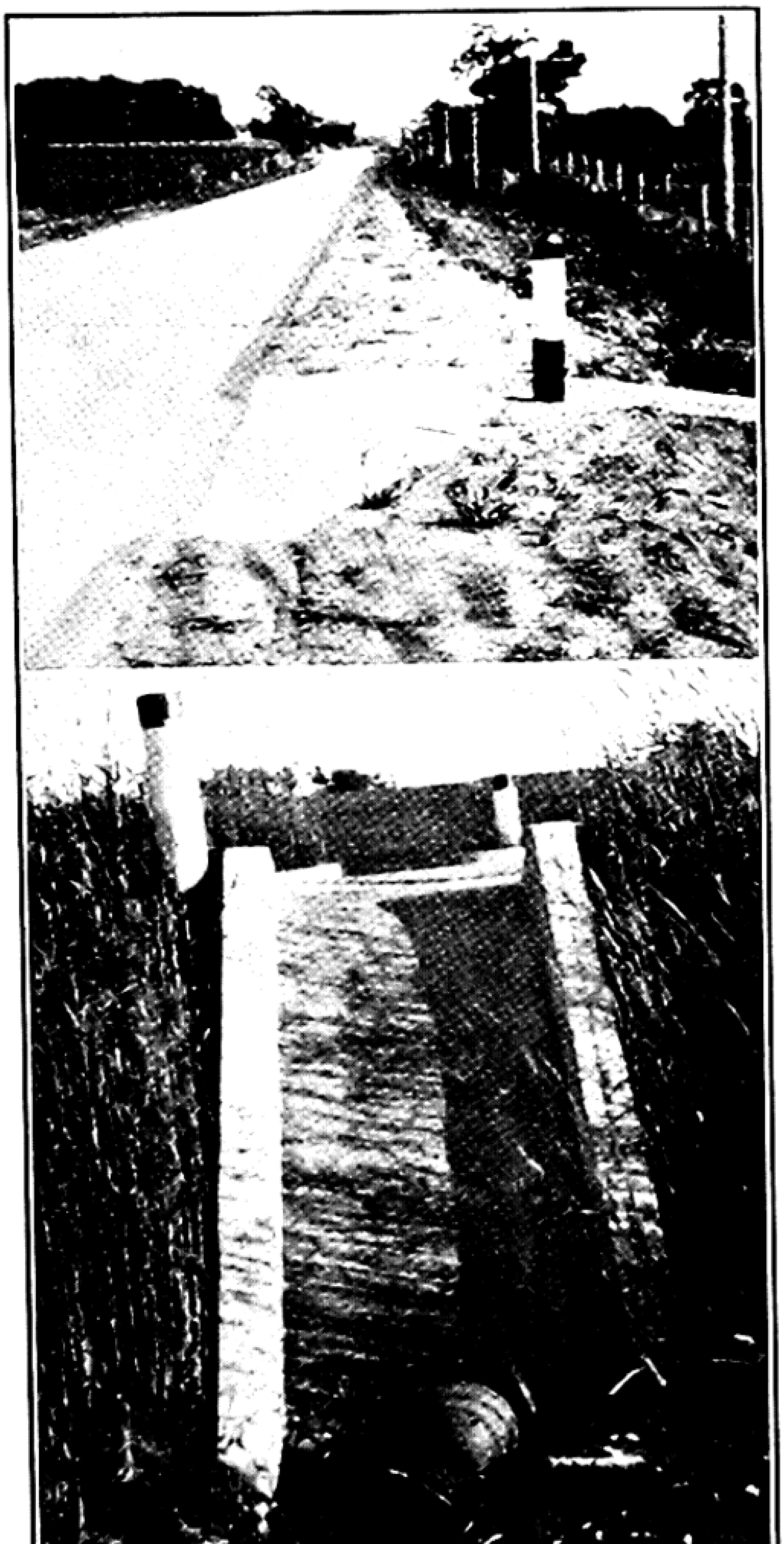


Fig. 89.—Type of gutter outlet and flume.

Storm-water Flumes.—When surface water is carried on the pavement slab for convenient disposal it may be led from the pavement slab to an open ditch at the side of the road, or it may be carried to an underground tile drain.

When the water is to be disposed of in open ditch at the side of the road, it is customary to employ an open flume from the pavement slab to the ditch. These flumes are usually made of concrete, and the design is such that where the flume crosses

the shoulder it will be possible for a vehicle to cross the flume without incurring danger. The shape of the flume is illustrated in Fig. 89 which shows the particular form desirable to lead the water on the down grade directly into the flume rather than having it flow past. Where flumes are used on relatively flat grades it is unnecessary to provide other than a symmetrical slightly enlarged section for the water to enter.

Inlets.—Although storm water is carried in open channels along the rural highways as a general rule, there are cases in which the water must be led to an underground tile line. In municipal pavements it is customary to carry all the storm water in an underground tile system, and consequently inlets must be employed to get the water from the pavement to the underground channel. A general observation may be made that any inlet is a source of difficulty because the openings must be small enough to insure against pedestrians, especially children, or wandering animals getting their feet caught in the grating. If the mesh is small enough to be safe, it will be small enough to clog up with debris carried by storm water. The tendency to clog is accentuated if the water tends to flow vertically through the grating. On the other hand, automatic self-clearing is in a measure provided if the storm water must make a turn and flow through a slot instead of a grating. In consequence we find that there are two types of inlets employed, the curb inlet which is a slot in the face of the curb, and the gutter inlet which is a grating in the gutter. These are illustrated in Fig. 55. Where gratings are used in rural highway construction it is important to insure smooth riding at the grating. Therefore, there should be no break in the curb or gutter grade line at the grating. In municipal practice the gutter may be depressed slightly at the inlet grating, but the term "slightly" should be interpreted to mean not more than ½ in., sloped back for 2 or 3 ft. along the gutter line, with no slope in the pavement slab itself.

Aggregates for Concrete Pavements.—The old conception of concrete was that it consisted of four ingredients: coarse aggregate, fine aggregate, cement, and water. The modern conception of concrete is that it consists of two ingredients: aggregate and cement paste. Despite this conception the commercial production of aggregates makes it necessary to consider the mineral aggregate to be made up of two parts, the coarse and the fine. Where broken stone constitutes the coarse, and natural sand

the fine, aggregate, these materials are produced in separate plants, and the constructor must use such a combination of coarse and fine materials as will give him the desired strength at the lowest cost. Where natural sand-and-gravel mixtures are produced for the aggregate it is physically possible to produce a combined coarse and fine aggregate of the desired grading; but on account of the tendency to segregation in handling, it has proved to be more satisfactory to separate these aggregates into coarse and fine and recombine them at the mixing plant. Consequently, although concrete itself consists of combined aggregates, the aggregate itself must of necessity ordinarily be handled as coarse aggregate and fine aggregate.

The principles of particle-size distribution that were discussed on page 87 apply to concrete aggregates in that the best strength and durability will ordinarily be obtained with concrete in which the aggregates are so graded as to approach a particular Talbot curve for maximum density of the general form of those in Figs. 72 and 75 but with constants of Equation (1), page 87, selected to fit the particular type of aggregate to be employed for the concrete. The ratio of water to cement has so powerful an influence upon the strength of the concrete, however, that more attention is paid to the water-cement ratio than to the grading of the mineral aggregate. The engineer should, however, take both factors into account and, by laboratory studies on the available materials, determine the proportion of coarse and fine aggregates of the kinds commercially available that will produce concrete of the desired strength at the lowest cost.

Requirements of the aggregate in general may be emphasized by a discussion on several properties that are deemed important.

The aggregate should be sound, by which is meant that the effect of repeated freezing and thawing, together with absorption of water, does not cause the aggregate to soften sufficiently so that it will be unable to resist the wear to traffic. In general, the concrete road surface is subjected to the wear of rubber-tired traffic, and such investigations as have had to do with this particular factor seem to indicate that the wear of rubber tires on concrete-road surfaces is negligible. It is true that in certain areas repeated formation of ice or snow layers followed by thawing brings about a condition under which a good deal of the traffic must travel with chains on the tires and that under such conditions there will be appreciable wear on the concrete.

Aggregates in which there are particles of chert or shale are to be avoided, because both of these materials absorb water and, upon freezing, increase in volume, thus introducing internal stress in the concrete; and if the particles lie near the surface, they will inevitably rupture the surface, and the result will be a pitted pavement. It is also highly desirable to select stone that breaks with a granular fracture rather than a glassy one. Occasionally stones are encountered the surface of which at the fracture is vitreous or glassy in appearance, and such stones do not bond well in concrete.

By grading is meant the proportion of the several sizes in the combined coarse and fine aggregate. The maximum size should not be greater than about 2½ in., although stone having a maximum size of 3 in. has been employed in concrete pavement with entire success. The minimum size is that which will pass a 50-mesh sieve; of this there should not be more than 5 or 6 per cent. Between these two sizes the proportion of the several sizes should be such as to insure that when the particle-size distribution curve is platted it will be definitely concave upward, and the more nearly it approaches the curve of maximum density the better.

Aggregates produced from natural gravel deposits may contain appreciable quantities of shale or clay. This sort of material should be removed by washing so that the resulting aggregate does not contain more than 2 or 3 per cent of clay and silt, and it is especially important to avoid materials in which the clay is in the form of a coating on the particles, because such materials do not bond well in concrete. If aggregates contain an excessive amount, say more than 5 per cent, of very fine particles, these tend to rise to the surface during the finishing of the concrete and segregate in the upper ⅛ in. or so of the surface, thus producing a layer that does not withstand wear and gradually peels off the surface under traffic.

Broken-stone Coarse Aggregate.—Broken limestone, quartzite, granite, and the several varieties of traprock all are employed as coarse aggregate for concrete pavement. The minimum requirement for the quality of the broken stone is that it shall have a French coefficient[1] of wear of not less than 6. But for trunk highways in general, stone having a quality indicated by a French coefficient of at least 7 should be employed if it can

[1] Reference to A.S.T.M. test, p. 313.

be obtained. In those areas where there is to be expected a considerable amount of vehicle traffic with antiskid chains, the French coefficient should certainly be not less than 7 and preferably at least 8. The stone should be well graded from coarse to fine, as has already been discussed, in order to aid in securing a workable mixture with a low water-cement ratio. Rock that breaks with a slivery fracture and results in a product containing many flat and elongated pieces is objectionable because such pieces float to the surface and fracture under traffic, resulting in a pitted surface.

Broken-slag Coarse Aggregate.—Broken slag has had a limited use as coarse aggregate for concrete pavement. Where the slag is of the proper quality, there is no reason why it should not be employed if it is economical. Since slag is a by-product of blast-furnace operation, there is likely to be a great deal of variation in the quality of the material available for crushing; and, in general, slag that is handled promiscuously when dumped for cooling is likely to be too porous and soft for use as a coarse aggregate.

Sandstone Coarse Aggregate.—Although sandstone blocks have been used extensively in the past as the wearing surface of pavement and have been quite satisfactory because of the anti-skid type of surface produced, broken sandstone has not been used extensively as the coarse aggregate for concrete. There is no reason why sandstone of the proper quality and properly prepared should not be employed as coarse aggregate.

Gravel Coarse Aggregate.—Pebbles screened from bank gravel have been extensively used as coarse aggregate for road concrete. Aggregate of this type usually contains a considerable percentage of angular material produced by crushing the oversized pebbles from the deposits. The requirement for quality is that the aggregate should have a percentage of wear of not to exceed 18 when tested according to the special abrasion tests for gravel.[1] It is also important that the gravel be free from mud balls and from any considerable percentage, say more than 3, of shale, shell, or other material that is materially softer than the run of gravel pebbles. Generally speaking, concrete can be produced with gravel with a somewhat lower water-cement ratio than with broken stone and still insure a mixture that is workable.

[1] Reference to A.S T.M. test, p. 313.

Fine Aggregate.—The fine aggregate for concrete road construction is ordinarily sand from natural deposits. The requirements for the fine aggregate are that it shall be free of deleterious material such as clay balls, shale particles, mica flakes, and organic matter. The quantity of clay in the sand is determined by the elutriation test.[1] The quantity of mica is determined by carefully sorting the mica flakes from a small sample of the sand and determining the percentage, which should not exceed 2.

It is advisable to investigate in the laboratory the available aggregates by combining the sand proposed to be used with the coarse aggregate in various proportions until the most advantageous particle-size distribution of the combined aggregate is obtained. Frequently it is possible to affect materially the cost of the concrete by such a laboratory study of the available coarse and fine aggregates to determine the materials that will combine most advantageously.

The quality of the fine aggregate is still further checked by determining the strength of mortar made with the proposed aggregate in comparison with the strength of mortar made from standard Ottawa sand. Certainly in no case should fine aggregate be employed in which the mortar strength as shown by this test is less than the mortar strength of standard Ottawa sand. Most good concrete sands will have mortar strength considerably in excess of the mortar strength made with Ottawa sand of the same percentage of cement and water-cement ratio.

Concrete Mixtures.—Concrete road slabs are designed on the basis of an assumed strength of concrete, and the first consideration in the design of the concrete mixture itself is to make sure that a concrete is produced that will have strength at least equal to that assumed in the design. The prevailing practice in 1939 is to design the road on the assumption that the safe working flexural modulus of the concrete will be at least 350 at the age of 28 days, which means that the flexural modulus of rupture of the concrete should be at least 700 at that age. In the mixtures that have been used since 1935, the design of the mixture has been such as to produce concrete ranging in 28-day strength from as low as 600 p.s.i. to a flexural modulus of rupture of 1,000 p.s.i. Probably the average has not been far from 700 p.s.i. If this is interpreted in terms of the compressive strength of the concrete,

[1] Reference to A.S.T.M. test, p. 314.

it means that the average compressive strength of the concrete used in standard design has been about 4,000 p.s.i. at the age of 28 days.

Although standard specifications frequently mention the proportions of coarse and fine aggregate to be used with a cubic foot of cement, this is generally to be considered only as a guide, and the exact proportions are developed by the laboratory tests to establish the most economical combination of the available fine and coarse aggregate that will produce concrete of the required strength. If arbitrary proportions are to be used with standard aggregates, the mixture most widely employed would be one part cement, two parts fine aggregate, and three or three and a quarter parts of coarse aggregate (written 1–2–3 or 3¼ concrete). The aggregates are proportioned by weight, and the water measured. It is recognized that the critical element in a mixture of concrete is the water and that the best concrete will be that which is produced with the lowest water-cement ratio consistent with a workable mixture. For machine finishing such as is employed on concrete roads, a slump[1] of about 1 or 1½ in. is ample; and if the aggregates are properly proportioned, such a mixture can be produced with somewhat less than 6 gal. of water per cubic foot of cement. No mixture should be employed that requires more than 6 gal. of water per cubic foot of cement; and if a workable mixture can be produced with 5 or 5½ gal. of water per bag of cement, that is highly desirable.

CONSTRUCTION DETAILS

The construction of concrete road surfaces has been characterized by a continual evolution of new methods and new devices for lowering costs and increasing production. Many variations in practice are to be observed, and new schemes for performing minor operations will be noted each year on the work of progressive constructors.

Finish Grading.—The effort to eliminate irregular settlement of the subgrade and to secure a supporting layer of soil of uniform composition and a particle-size distribution favorable to high supporting strength requires that the entire area of the subgrade for a depth of 6 in. to 1 ft. be manipulated in the process of finishing the grading operations. The methods whereby this supporting layer may be brought to the desired composition have

[1] Reference to slump test, p. 315.

been discussed elsewhere; but in the final preparation of the subgrade it is assumed that the finish grading operations have been completed according to an appropriate plan and that there

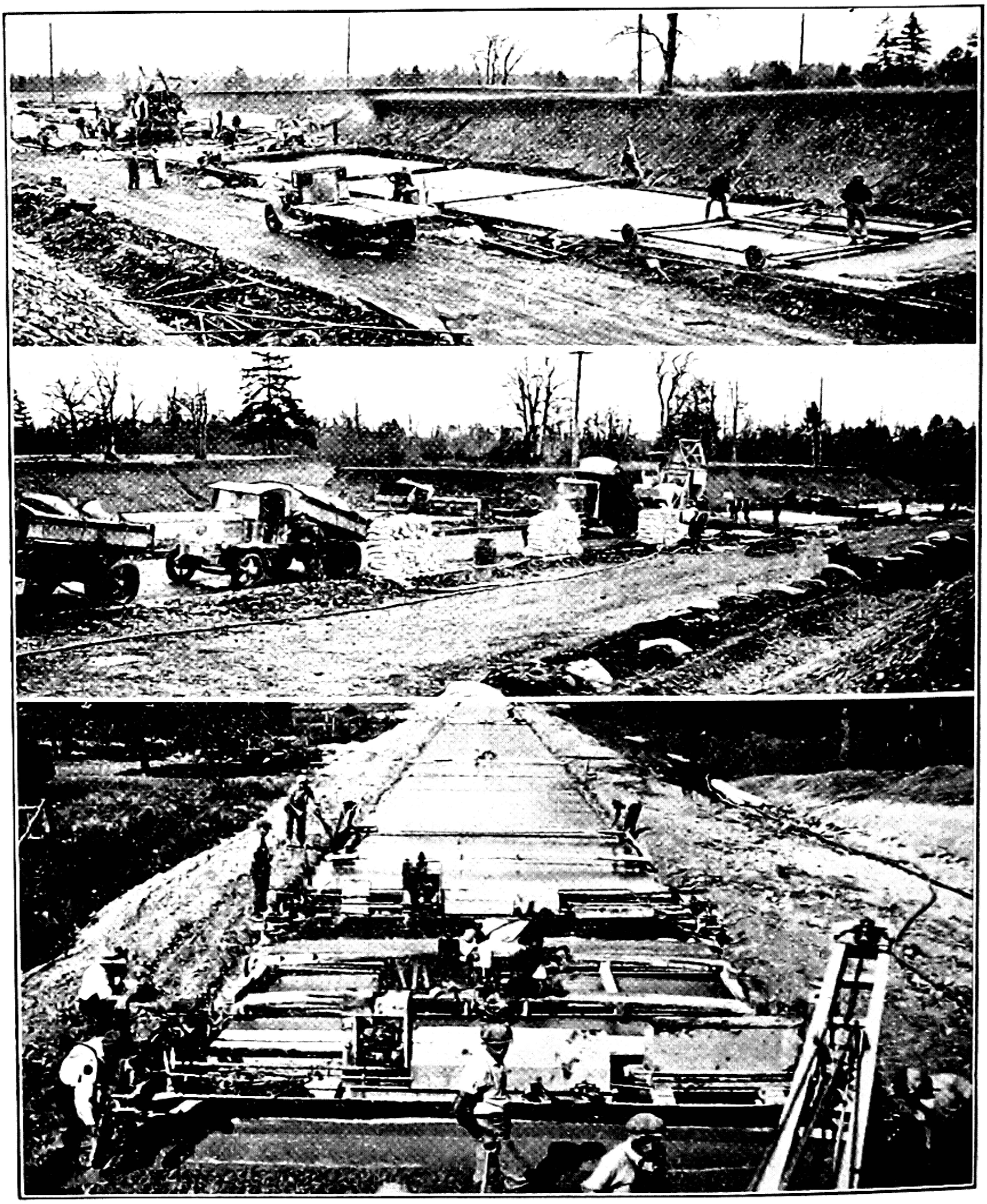


Fig. 90.—Construction of concrete pavements.

remains only the task of bringing the surface of the subgrade to the shape and height prescribed so that the layer of concrete placed thereon will be of the correct thickness.

Ballast Layer.—The use of a ballast layer under the concrete slab has already been mentioned, but such a type of construction is feasible only where suitable materials for the ballast course can be secured at low cost. In many regions the cost of the ballast course is prohibitive, and the design of the road slab is predicated upon its being placed directly on the subgrade soil. If the ballast layer is adopted, the final shaping of the subgrade consists merely in shaping the ballast layer which can be accomplished with templates resting on the side forms and drawn by the mixer so that the subgrade is shaped immediately ahead of the area on which concrete is being placed.

Preparation of Subgrade.—The subgrade is the portion of the roadway upon which the concrete slab is placed; and since it is imperative to secure as nearly as practicable the slab thickness provided for by the plans, it is shaped with considerable care. After the rough grading has been completed, the side forms are set and are then available as guides for the completion of the final grading. For this final grading a subgrading machine is used unless the cross-section is unsymmetrical, in which case hand tools must be used to dress down the high places left by the rough grading operations. The special shape required at the edge of the subgrade for slabs with thickened edges is secured by the use of hand tools. The accuracy of the subgrade is checked by a template which rests on the side forms.

During the final dressing of the subgrade it is repeatedly rolled to insure uniform compression and eliminate soft areas which would become depressions during the period when trucks use the subgrade in delivering materials to the mixer. Some constructors keep a light roller working on the subgrade continuously to prevent ruts or other uneven places from developing. As a final check on the accuracy of the subgrade, a heavy template with an iron-shod cutting edge is drawn along the forms just behind the mixer to shave off any high areas that may have escaped notice up to that time.

Subgrades for wide pavements are shaped by the foregoing methods, lane by lane, except in municipal work where warped surfaces must frequently be used. These are brought roughly to the desired form with a blade grader and finished by hand methods.

Dry subgrades are thoroughly sprinkled before the concrete is placed so that the soil will not draw water from the freshly placed

concrete. Very porous soils that are dry may take so much water from the concrete that shrinkage cracks will form during the period when the concrete is setting. If sprinkling the subgrade does not prevent the formation of these cracks, a layer of light roofing paper may be placed on it.[1]

Forms.—The forms placed at the sides of the slab to retain the green concrete are also required to carry the finishing machine or the strike board used for shaping the surface of the concrete. Since these devices are of considerable weight, it is important to use rigid metal forms which are securely staked in position, properly adjusted to line and grade, and firmly bedded so that they will not be displaced or depressed by the operation of finishing the concrete. In municipal practice, wooden forms are still employed to a limited extent; and if they are to be satisfactory, they must be of 4-in. dressed plank and must be carefully staked and braced to line and grade.

Water Supply.—The water supply for concrete work may be obtained by connection to a municipal water supply, in which case the connection is made to the fire hydrants along the street, usually by means of hose. If a municipal water supply is utilized for rural highway construction, a pipe line is laid from the city main along the route of the proposed improvement, with cocks at frequent intervals from which a hose is connected to the mixer.

In many cases the water supply for rural highway construction must be obtained from streams along the highway, and the pipe line is laid from these streams in the usual manner. At the source of supply, a motor- or gasoline-engine-driven pump of the triplex variety is employed for furnishing the water supply. It is not uncommon to use duplicate water supply equipment so as to avoid a possible shutdown on account of pump difficulties. Pipe lines are laid on top of the ground with provisions at intervals for equalizing the pressure. The size of pipe varies from 2 to 4 in., depending upon the length of the line, but the tendency is toward a large size so as to limit friction losses in the pipe to an amount that will not interfere with the delivery of an adequate supply of water.

Proportioning Plants.—The proportioning plant consists of elevated bins to which the aggregates are delivered by means of cranes or bucket elevators. The aggregates are drawn from

[1] Crum, R. W., "Hair Cracks in Concrete Pavements," *Eng. and Contracting*, Vol. 64, No. 4, p. 791, Oct. 7, 1925.

the overhead bin into the weighing device for proportioning the several aggregates. Proportioning plants are of many varieties and may be constructed of timber or of steel. It is customary to set up the proportioning plant at the railway siding where the aggregates are delivered or at a local quarry or gravel plant.

The equipment required for measuring the aggregates for batching are incorporated in the plant in such a manner that the materials flow by gravity from the storage bins to the weighing device and then to the trucks. If the concrete is mixed at the proportioning plant, the mixer is mounted in the plant at such an elevation that the materials flow by gravity from the storage bins to the weighing device, then to the mixer, and finally into the trucks.

Dry-batch Method of Proportioning.—The dry-batch method of proportioning is one in which the aggregates are weighed into the truck; the cement added; and the load hauled to the mixer, which travels on the subgrade. The aggregates are dumped into the mixer; the water added; and the concrete, when mixed, deposited directly upon the subgrade. This method of handling aggregates is probably more widely used than any other.

Wet-batch Methods of Placing Concrete.—In the wet-batch method of placing concrete the aggregates are proportioned and dumped directly into a mixer which is located at the proportioning plant. The concrete is there mixed, discharged into the transportation, and hauled as mixed concrete to the place where it is to be deposited. The loads are dumped upon the subgrade and finished in the usual manner. This method has been quite successfully employed on a number of jobs and is entirely satisfactory if the haul does not exceed about 45 min., but with the mixer-type truck the haul may be up to 2 hr., and the cost of such long hauls would be prohibitive. The wet-batch method is particularly adaptable to those locations where it is difficult to provide an adequate water supply along the road.

Mixing the Concrete.—The mixers employed for pavement construction will produce about 1 cu. yd. of concrete per batch. The materials are dumped into the loading skip which is then hoisted so that the aggregates will slide into the mixing drum. The water is introduced along with the dry aggregates, and the mixing continues for about 1 min. Although investigations have shown that well-mixed concrete can be produced in about 40 sec., yet with all the uncertainties involved in this operation

it is deemed wise to continue the mixing for the period of one full minute.[1] Upon the completion of the mixing, the concrete is chuted or conveyed by bucket to the place where it is to be deposited on the subgrade. The mixer sets the pace for all the construction operations, and the whole organization is built around it with a view to getting maximum production out of it. The mixers travel on the subgrade, generally on the crawler type of tread, and can therefore be used when the subgrade is slightly moist. Concrete placing cannot be carried on if the subgrade is very wet, because the hauling will disturb the latter to such an extent as to interfere with securing the proper thickness of slab.

Machine Finishing.—The first finishing operation is performed by means of a machine that strikes the concrete to the required cross-section, at the same time screeding it and working the surplus water out of it. The working element of the finishing machine consists of a heavy metal screed which rests on the side forms and oscillates transversely, at the same time moving forward along the forms at a slow rate. The finisher is operated by a gasoline engine carried by the frame of the machine, and the whole assembly rides on the side forms on flanged wheels. Some finishers have two screeds for striking the concrete and sometimes a belt for final finishing, all operated as a single unit. After the first trip over the surface with the finishing machine the screeds are raised, the machine moved back, and the second finishing accomplished in exactly the same manner as the first. If the forms are rigid and set firmly to grade, this machine will provide a smooth, gritty surface of excellent texture. The final finishing is carried out by longitudinal floating. These longitudinal floats consist of a 2- by 8-in. plank, about 12 ft. long, spiked to the edge of a second plank to form a T-shaped float, with plow handles attached to each end. This special float is operated by two men who stand on bridges placed across the concrete slab. They float the surface with a longitudinal movement, thus insuring that all cross-ridges are worked out of the green concrete. After covering the width of the slab with the float, the bridges are moved forward along the side

[1] Harrison, J. L., "Effect of the Length of the Mixing Period on the Quality of Concrete Mixed in Standard Pavers," *Public Roads*, Vol. 9, No. 5, p. 93, July, 1928.

forms about half the length of the float, and the process repeated. This is continued throughout the length of the slab.

Water Curing.—The most common method of curing the concrete road slab is to sprinkle it lightly as soon as the concrete has set sufficiently to permit the light spray being used without forming pits in the surface. This light sprinkling is continued until the concrete has set sufficiently to permit covering the surface with a burlap mat. The burlap is kept wet until the concrete has hardened sufficiently to permit an earth covering. The earth covering is provided by shoveling on to the surface about 2 in. of loose earth from the shoulder. As soon as the earth is in place it is thoroughly wetted and is kept wet for a period of about 10 days.

In some cases the surface is cured by the method known as "ponding." Dikes of earth are placed across the concrete at intervals, and the surface flooded so that the concrete is covered with 2 or 3 in. of water, and this is replenished as it evaporates or soaks away so that the concrete is kept covered for a period of about 10 days.

Calcium-chloride Curing.—Calcium chloride in flake or granular form is also used to a limited extent as a curing medium. As soon as the cement takes initial set, the calcium chloride is sprinkled on the surface of the concrete in sufficient quantity to cover the entire surface with a thin layer, and as the calcium chloride takes up water, it melts down into a covering that seals the entire surface, keeping the surface moist and preventing the evaporation of water from the concrete. This method has been used quite widely and is acceptable if care is taken to secure an even distribution of the calcium chloride.

Bituminous Coating for Curing.—Bituminous coatings applied to the green concrete are also employed as a curing medium. The bituminous mixture in the form of a thin cut-back, or emulsion, is sprayed over the concrete while it is still green, thus sealing the surface so that water will not evaporate from the concrete. When the road is placed in service the thin layer of bituminous material gradually wears off the surface, leaving the concrete with the ordinary texture and a somewhat darkened color.

Finishing Methods for Wide Streets.—Streets that are so wide that the finishing machines cannot be used must of necessity be hand-finished, but wherever possible the pavement should be

built in lanes each of which is machine-finished. For streets up to 30 ft. in width it is practicable to use a strike board for shaping the green concrete. This strike board rests on the side form and is drawn along the form by laborers who employ a tamping and forward movement which strikes the surface and pushes the surplus concrete ahead. As soon as they have struck off the surface, the concrete is smoothed with hand floats, usually long-handled ones, which are operated from the side of the pavement. Finishing at contraction and expansion joints must be carried out from a bridge or from planks that rest on the green concrete. Wide streets are often laid in strips, each of which is finished and cured before the next parallel strip is laid, and often the design permits each strip to be machine finished. This method is particularly advantageous for streets more than 30 ft. in width. Longitudinal joints are provided to separate the several strips that are laid.

Curing Period.—Concrete surfaces are opened to traffic as soon as the concrete has attained the strength assumed in the design. This is determined by breaking test specimens fabricated from day to day from the concrete that is being placed and cured as the road slab is being cured. The specified strength is attained in 4 or 5 days in warm weather and in various longer periods in cool weather. In the absence of test specimens the concrete is cured for about 2 weeks in normal summer weather and in cool weather for a longer period, determined by the conditions.

Resurfacing Concrete Roads and Pavements.—If it becomes desirable to resurface a concrete road slab, a new concrete surface may be added, or the old concrete slab may be utilized as a base for some other type of surface. The existing conditions will in each individual case indicate which is preferable.

A number of concrete roads have been resurfaced with concrete, and there seems to be no question as to the stability of the resulting structure. The old surface is cleaned of any accumulation of foreign material, but no attempt is made to provide a bond between the new concrete and the old. Potholes in the old slab are first filled with concrete to restore the contour of the old surface. The new surface is then placed and finished exactly as though it were an ordinary concrete pavement. The concrete is of the quality already discussed, and the thickness is usually about 4 in., although surfaces as thin as 2 and 3 in. have been constructed successfully. Embedded steel in the form of mesh

or bars is placed in the concrete; and if the old surface was provided with expansion joints, such joints are placed in the new surface directly over the old ones. The concrete resurfacing is carried out in substantially the same manner as the construction of a new pavement previously described, and the finishing and curing operations are especially emphasized.

THE TECHNOLOGY OF PAVEMENT CONCRETE

The technology of pavement concrete consists in the application of the knowledge of concrete and concrete aggregates to the control of concrete fabrication for pavement construction. The subject will be presented herein only in outline to indicate the nature and scope of the laboratory and field tests required to control adequately the quality of the concrete of which pavements are made.

Sampling Materials for Testing.—The technique of materials testing has developed on the theory that the characteristics of a material supplied in commercial quantities can be determined by testing a small sample of that material. That theory is valid if the selected sample is truly representative of the product. Concrete aggregates are not required to meet close tolerances, but the samples selected for testing should fairly represent the average run of the product. Samples of stone, slag, gravel, sand, and stone blocks may be taken according to A.S.T.M. D75-22, 1936 B.S., Part II, p. 1092.

Tests of Aggregates.—Coarse aggregates are subjected to a number of tests for the purpose of appraising their suitability for pavement concrete or for determining the most economical mixture of aggregates for concrete of the desired properties.

1. *Wearing Quality.*—The wearing quality of coarse aggregates is determined by one of the abrasion tests. Crushed stone is tested by the Abrasion Test of Rock, A.S.T.M. D2-33, 1936 B.S., II, p. 1040, or by the use of the Los Angeles Test Machine, *Proc. A.S.T.M.*, Vol. 35, 1935, I, p. 350. Gravel is tested by the Tentative Abrasion Test of Gravel. A.S.T.M. D289-28T (T indicates tentative standard), 1936 T.S., p. 738.

2. *Grading (particle-size distribution) of aggregates* is determined by the Test for Sieve Analysis of Aggregates for Concrete. A.S.T.M. C41-36, 1936 B.S., II, p. 353.

3. *Absorption of Water.*—The absorption of mixing water by aggregates is determined by the Tentative Field Test for Absorption. A.S.T.M. C96-36, 1936 B.S., II, p. 332.

Absorption of water is determined in the laboratory by the Tentative Test for Absorption. A.S.T.M. C95-36, 1936 B.S., II, p. 330. The surface

moisture held by fine aggregates is determined by the Test for Surface Moisture. A.S.T.M. C70-30, 1936 B.S., II, p. 357.

4. *Deleterious Impurities.*—The presence of deleterious substances in aggregates is determined by subjecting the material in turn to one or more of the following tests.

a. Method of Testing Concrete Aggregates by Freezing and Thawing (Proposed Draft). *Proc. A.S.T.M.*, Vol. 32, 1932, I, p. 364.

b. Decantation Test for Clay and Silt in Gravel. A.S.T.M. D72-21, 1936 B.S., II, p. 1050.

c. Decantation Test for Sand and Other Fine Aggregates. A.S.T.M. D136-28, 1936 B.S., II, p. 1051.

d. Test for Organic Impurities in Sands for Concrete. A.S.T.M. C40-33, 1936 B.S., II, p. 350.

e. Soundness of Coarse Aggregates. A.S.T.M. C89-35T, 1936 T.S., p. 502.

f. Soundness of Fine Aggregates. A.S.T.M. C88-35T, 1936 T.S., p. 507.

5. *Quality of Fine Aggregates.*—The quality of fine aggregates is appraised on the basis of the tests mentioned in the foregoing sections and the determination of the mortar-making properties indicated by the Test for Structural Strength of Fine Aggregates, A.S.T.M. C87-36, 1936 B.S., II, p. 355.

6. Certain miscellaneous tests for aggregates are useful in the investigation of concrete mixtures.

a. Tentative Test for Amount of Material Finer than 200-Mesh. A.S.T.M. C117-35T, 1936 T.S., p. 488.

b. Tentative Specification for Commercial Sizes of Broken Stone and Broken Slag. A.S.T.M. D63-23T, 1936 T.S., p. 719.

c. Tentative Specifications for Commercial Sizes of Sand and Gravel. A.S.T.M. D64-20T, 1936 T.S., p. 713.

d. Tentative Specifications for Concrete Aggregates. A.S.T.M. C33-36T, 1936 T.S., p. 482.

e. Specific Gravity and Absorption of Aggregates: coarse, A.S.T.M., C127-36T, 1936 T.S., p. 511; fine, A.S.T.M. C128-36T, 1936 T.S., p. 514.

f. Unit Weight of Aggregate for Concrete. A.S.T.M. C29-27, 1936 B.S., II, p. 364.

g. Voids in Coarse Aggregate. A.S.T.M. Tentative Standard, C30-35T. 1936 T.S., p. 517.

h. Voids in Fine Aggregate. A.S.T.M. C30-22, 1936 B.S., II, p. 360.

i. Voids in Inundated Fine Aggregate. A.S.T.M. C69-30, 1936 B.S., II, p. 361.

7. *Tests of Concrete.*—A number of tests are available for checking the quality of the concrete as produced on the construction job and for miscellaneous materials used in placing concrete.

a. Field Determination of the Constituents of Fresh Concrete (Proposed Draft). *Proc. A.S.T.M.*, Vol. 31, 1931, Part I, p. 383.

b. Compression Tests of Concrete. A.S.T.M. C39-33, 1936 B.S., II, p. 342; C116-36, 1936 B.S., II, p. 348; Specimens for C31-33, 1936 B.S., II, p. 337.

c. Flexure Tests of Concrete. A.S.T.M. Tentative Standard, C78-36T, T.S., p. 493.

d. Cement Content of Hardened Concrete. C85-36, 1936 B.S., II, p. 334.

e. Specification for Concrete for Pavements. A.S.T.M. Tentative Standard, D366-33T, 1936 T.S., p. 694.

f. Consistency of Portland Cement Concrete. A.S.T.M. Tentative Standard D138-32T, 1936 T.S., p. 741.

g. Sodium Silicate for Curing Concrete. A.S.T.M. C111-36, 1936 B.S., II, p. 329.

h. Surface Application of Calcium Chloride for Curing Cement. A.S.T.M. C83-36, 1936 B.S., II, p. 327.

i. Wet Coverings for Curing Concrete. A.S.T.M. C84-36, 1936 B.S., II, p. 328.

j. Bituminous Coverings for Curing Concrete. A.S.T.M. C81-36, 1936 B.S., II, p. 324.

CHAPTER XIII

VITRIFIED BRICK PAVEMENTS

The brick pavement is one of the older standard types in use in the United States, particularly for city streets. Its construction reached a high state of excellence at a time when horse-drawn traffic with its metal-tired wheels constituted the most severe test of pavement durability. When traffic shifted to the types of vehicles with rubber tires, with attendant greater load concentration, road builders clung tenaciously to the old standards of design and for a period the brick pavement lost standing in comparison with some of the other types. In recent years a serious attempt has been made to bring the design of the brick pavement into harmony with the requirements for durability under present-day traffic and to regain the lost prestige. It seems likely that this most excellent type will assume increasing importance as time passes.[1]

Some typical cross-sections for brick pavements are shown in Fig. 91. These illustrate the composition of the pavement, but the detailed dimensions are a matter of design that is dependent upon the traffic to be served in any specific location.

PAVING BRICK

Paving brick are manufactured from shale by what is known as the stiff-mud process. The shale is ground and puddled in a pug mill with enough water to produce a very stiff mud. The mud is forced through dies under considerable pressure and the column of clay that is extruded is cut into blocks of the desired size.

The process of manufacture of paving brick tends to introduce certain physical defects which have no particular significance unless they become so pronounced as to interfere with the proper laying of the brick or impair the strength. Certain of these defects are cause for rejection under most specifications.

[1] "Report of the Investigation of Paving and General Highway Conditions," by the Engineering Commission appointed by the National Paving Brick Manufacturers' Association, Washington, D. C., 1929.

Laminations.—In the process of puddling the clay there is a tendency for layers to develop in the mass, and if the clay is not of the best quality these layers will not wholly weld together.

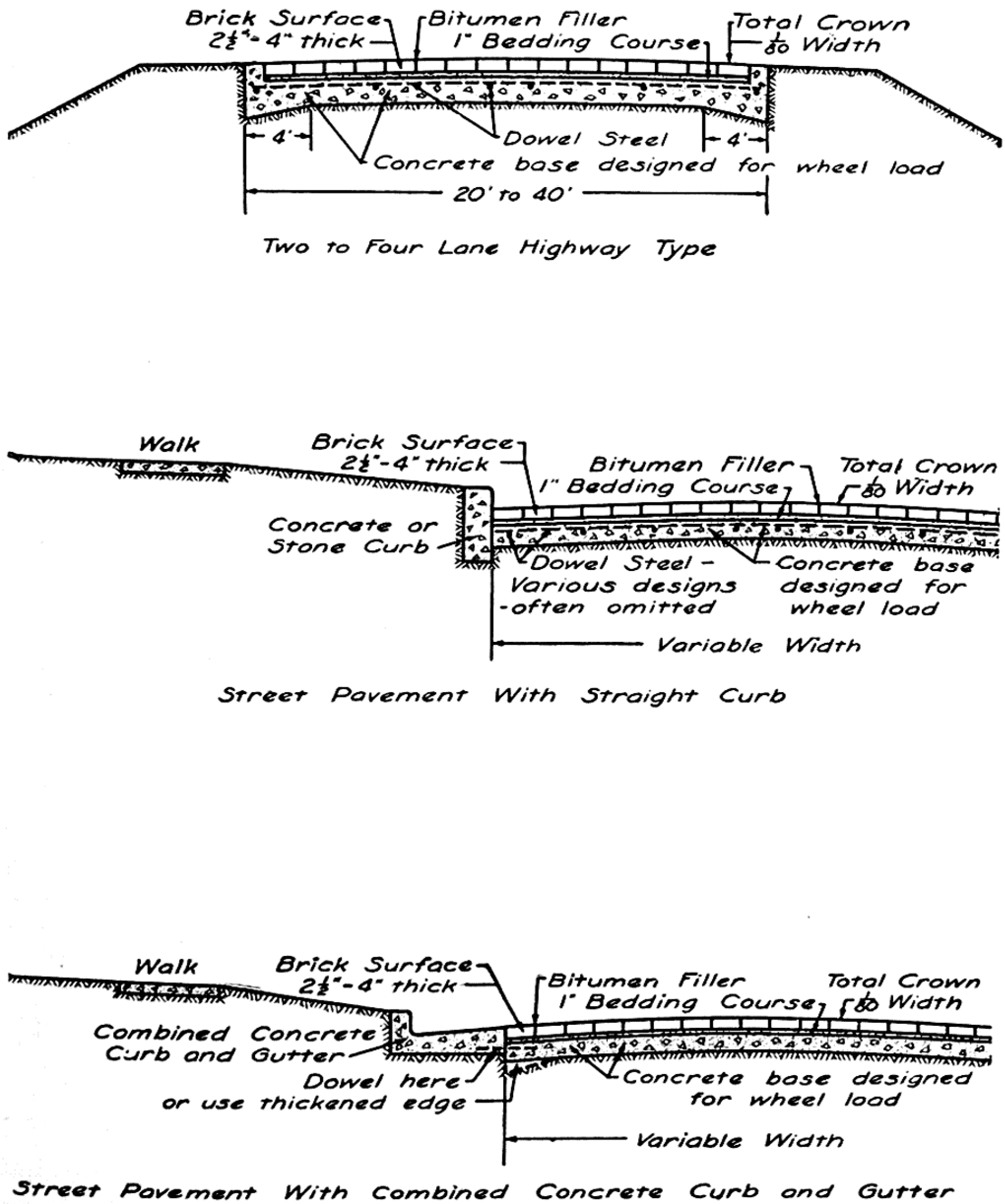


FIG. 91.—Typical cross-sections for brick roads and pavements.

This is evidenced by cleavage planes known as "laminations" which have no particular significance unless so marked that they indicate structural weakness in the brick.

Kiln Marks.—In the kilns, the brick are stacked in such a way that each brick will ordinarily have three good faces. The fourth face may be slightly distorted in the burning and may bear indentations from the brick upon which it rested, or that rested upon it. These kiln marks are not objectionable unless they are so pronounced that the brick will not lie evenly in the pavement.

Checks.—If the brick cool too rapidly they check because of contraction. Small checks are not uncommon but have no significance. Large checks are the cause for rejection.

KINDS OF PAVING BRICK

Several varieties and sizes of paving brick are produced, some of which are patented or trademarked and others of which are produced as stock types by a number of factories. There were at one time more than fifty sizes and varieties of brick being manufactured in the United States, but in recent years these have been reduced to the small number included in the present national standard sizes and types, which are generally called paving "blocks."

United States Standard Paving Bricks.—The following types and sizes of paving brick have been adopted by manufacturers and users as United States standards:

TABLE XXII.—SIZES AND VARIETIES OF PAVING BRICK

Variety	Width, inches	Depth, inches	Length, inches
Plain wire cut (vertical-fiber lugless)	2½	4	8½
	3	4	8½
	3½	4	8½
Repressed lug brick and wire-cut lug brick (Dunn type)	4	3½	8½

Repressed Vitrified Brick.—The repressed brick is first molded slightly thicker than the finished brick is to be, and without spacing lugs. It is then repressed in a mold that puts raised lugs on one face of the brick. The lugs serve to separate uniformly the brick in the pavement so that the filler will flow freely into the joint between courses.

It is generally recognized that repressing has a tendency to weaken the structure of the brick, especially with some kinds of clay. Since practically all paving brick are manufactured by the stiff-mud process, incipient laminations are inevitable, and these may be made more pronounced by the repressing process.

Non-repressed Brick.—In order to eliminate any possible objectionable effect of repressing and to cheapen the process of manufacture, several types of brick are now made by processes which avoid repressing. The lugs for spacing the brick in the pavement are secured by various means, and the differences in the processes are chiefly differences in the method of securing the lugs.

Wire-cut Lugs.—In the Dunn process the column of clay is cut the long way of the brick, *i.e.*, the brick are "side cut," and the cutting wire is guided so that it cuts spacing lugs on the brick. The vertical faces of the brick are therefore the cut faces, and being rough afford a surface to which the filler adheres readily. It will also be noted that the edges of the brick which form the upper edge of the transverse joint between the rows are square.

Vertical-fiber Brick.—The term "vertical-fiber brick" is a trade designation for paving brick that are manufactured in such a way that the laminations, if any exist, will be perpendicular to the surface when the brick is laid in the pavement. This type of brick is wire cut, but instead of the wire-cut face being vertical in the pavement it is the top or wearing face of the brick. To secure lugs, ridges or beads are molded on one edge of the brick. These beads are vertical when the brick are laid and extend entirely across the brick. This type of brick also has a square edge at the joint between rows.

Still another type of non-repressed brick is provided with spacing lugs consisting of four raised knobs about ½ in. in diameter. These are molded on the brick as the clay comes through the dies, by a special device attached to the die.

Standard Lugless Brick.—Manufacturers have recently adopted a standard lugless paving brick. One end of the brick is slightly convex to serve as an end spacer. The brick may be laid either for a 4-in. wearing surface or for a 3-in. If laid for a 3-in. surface it is a "vertical-fiber-lugless" brick and it is generally used as such.

The kiln marks and other slight inequalities of the brick are assumed to prevent adjacent brick from being placed so closely

together that the filler cannot flow into the space between the brick. This type has been very widely used and seems to be wholly satisfactory.

Recently this type of brick has been manufactured to lay a wearing surface 2½ in. thick and there is every reason to expect this size to be generally used for locations where individual loads are not excessive.[1]

PHYSICAL CHARACTERISTICS OF PAVING BRICK

The suitability of brick for use in pavements is determined by laboratory tests to determine ability to resist wear, to carry loads, and to withstand the deteriorating influences of the elements, and by visual inspection to detect physical defects such as laminations, kiln marks, checks, and brick chipped or cracked in the handling.

Wearing Qualities.—The wearing properties of brick are determined by the A.S.T.M. Standard Rattler Test. The limits placed on the loss during the rattler test will vary with the class of service demanded. The uniformity of the brick will be shown by the loss of individual brick, and it is therefore advisable in many instances to specify both the average loss of the 10 bricks and the maximum permissible loss for an individual brick. The data in Table XXIII indicate in a general way the limits that should be put on these two amounts.[2]

Table XXIII.—Limits for Rattler Loss of Paving Brick

Size, inches	Repressed brick, percentage loss		Wire-cut brick, percentage loss	
	Average	Maximum	Average	Maximum
3½ by 4 by 8½	22	24	23	25
3 by 4 by 8½	24	26	25	27
2½ by 4 by 8½	26	28	27	29

The quality of brick that should be used in any case is a matter that must be determined after a careful analysis of the traffic conditions and the foregoing table shows what is generally required for heavy-traffic pavements.

[1] Teller, L. W., and J. R. Pauls, "Thin Brick Pavements Studied," *Public Roads*, Vol. 7, No. 7, p. 129, September, 1926.

[2] Specification of Ohio Highway Department, 1929, p. 140.

Effect of Absorption.—The absorption test indicates the density and, in a measure, the degree of vitrification of the brick. These facts are also shown by the rattler test, and the absorption test is rarely specified for paving brick. When both the average and maximum loss during the rattler test are specified, it is superfluous to include also the absorption test.

Cross-breaking Test.—The cross-breaking test is rarely specified for paving brick except when brick other than standard blocks are used. As a matter of experience, brick rarely fail by cross-breaking or crushing. The test is useful in judging the strength of the 2½-in. brick now coming into use. The test is particularly applicable to vertical fiber brick made from clays that have a tendency to laminate.

Regularity of Shape.—The paving block must be sufficiently regular in shape to lie evenly in the pavement and form a smooth surface. For that reason the brick must be reasonably straight and have one good face. Warped, twisted, or spalled brick cannot be used.

Kiln marks are found on many good brick, but if these are not too deep and the brick has one good face it may be used.

Checks that are serious enough to indicate careless burning, too rapid cooling, or poor material, are a cause for rejection of the brick.

Laminations, as has already been explained, are an indication of poor quality, and if serious are cause for rejection of the brick. This factor of itself rarely causes rejection but in conjunction with other defects in some doubtful cases may be the one that determines the rejection.

Summary of Requirements for Paving Blocks.—The following abstract indicates the usual requirement for quality of paving brick for heavy traffic conditions.[1]

a. *Quality.*—These brick shall be wire-cut-lug or plain wire-cut brick with lugs, thoroughly annealed, tough, durable, non-absorptive, and evenly burned. When broken they shall show a dense stone-like body, free from lime, air pockets, cracks or marked laminations. Kiln-marks shall not exceed one-eighth (⅛) of an inch in depth and the wearing surface shall show only slight kiln-marks. Only one kind of brick shall be used on a continuous section or roadway.

b. *Size.*—Wire-cut-lug brick or plain wire-cut brick with lugs shall be three and one-half (3½) inches in width, eight and one-half (8½)

[1] Specifications, Pennsylvania Highway Department, 1929, p. 133.

inches in length, and three or three and one-half (3 or 3½) inches in depth, as may be specified on the drawings. The brick shall be provided with not less than two (2) projections or lugs on one (1) side, which shall extend from the body of the brick not more than one-quarter (¼) nor less than one-eighth (⅛) of an inch.

Brick shall not vary from the above dimensions more than one-eighth (⅛) of an inch in width or depth, nor more than one-half (½) of an inch in length.

The brick shall have square edges and the ends of the brick shall have a bulge of at least one-sixteenth (1/16) of an inch. The name of the brick or manufacturer if formed on the brick, shall be made by means of recessed letters.

c. Resistance to Abrasion.—The average abrasion loss on any kiln, pile, carload, or other lot of brick shall be determined by the rattler test, and the brick shall be of uniform quality as hereinafter specified.

For brick three and one-half (3½) inches in depth, and of length and breadth previously specified, the abrasion loss shall not exceed twenty-six (26) per cent by weight; for brick three (3) inches in depth and of length and breadth previously specified, the abrasion loss shall not exceed twenty-six and five-tenths (26.5) per cent by weight.

FOUNDATION COURSES

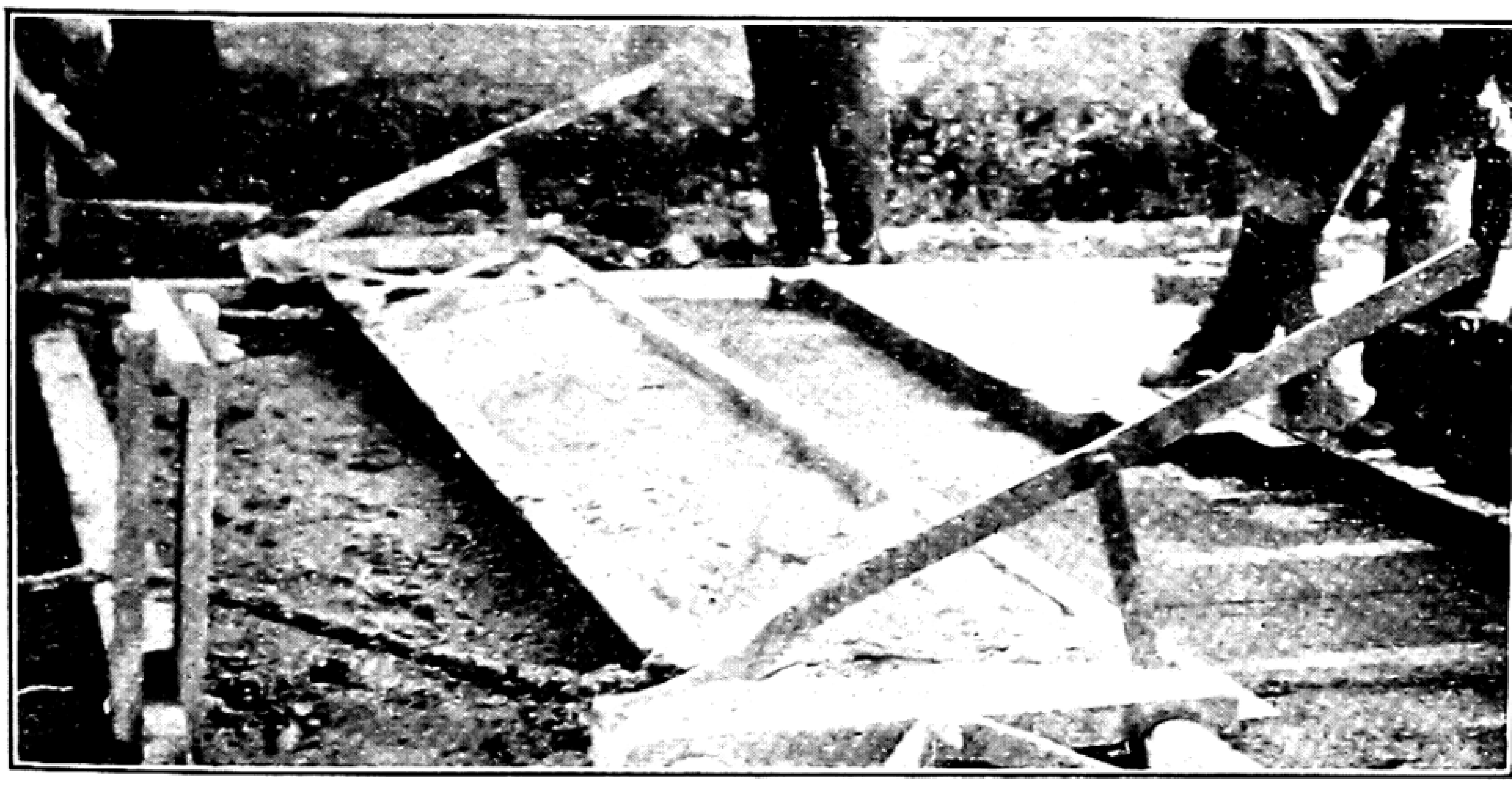
While it is sometimes economical to employ some other material, the concrete base is the predominant type for the brick pavement as well as for many other kinds of pavements.

There has been much discussion of the question of the thickness required for the concrete foundation for the brick pavement, and of the quality of the concrete. The following is based on what appears at the present time to represent the best knowledge of the subject:

Concrete Base for Bituminous-filled Brick Surface.—It is assumed that the brick surface adds nothing to the flexural strength supplied by the concrete foundation. Since the concrete slab will have construction joints in it, and will crack more or less on account of temperature effects, the critical condition is reached when a load is placed over a corner formed by the intersection of two cracks, or a crack and the edge of the base. It is possible that the bricks themselves will span the crack and that under traffic loads in the average case the concrete must deflect alike on both sides of a crack. If that assumption is correct the formulas developed for concrete road slabs with embedded steel may be applied to the design of the base for a brick pavement.

It appears to be much better engineering, however, either to design the base by the formulas for plain concrete road slabs or to use steel in the base. The choice is a matter of cost to a large extent.

In general the concrete foundation slab for a brick pavement presents a design problem involving an estimate of the probable load and impact allowance, strength of the concrete that will be used in the slab, and whether edge loads are to be expected. There is evidence to indicate that dowel steel and expansion joints are desirable for the concrete slab that is to be used as a base course for a brick pavement.



Courtesy of R. L. Bell.

FIG. 92.—Special template for monolithic brick road construction.

Many thousands of square yards of brick pavement are in use where the plain concrete base is only 5 or 6 in. thick, and the concrete mixed in about the proportion 1–2½–5, and their stability can be explained only on the theory that the brick span the cracks and joints and thereby affect the deflection under wheel loads, and that these pavements have never carried a volume of present-day truck traffic.

It is well to recognize the fact that the trend of weight of traffic units is upward and that the base thickness should be liberal because of the probable long life of the wearing surface and the heavier loads that will be likely to use the pavement eventually.

Concrete Base for Grout-filled Brick Surface.—In this type of construction the brick surface certainly adds to the flexural strength afforded by the concrete base, especially if the mortar

bedding course is used. The exact allowance to make for the strength added by the wearing surface has not been determined, but from the information available it is safe to compute the thickness required for the base without considering the strengthening effect of the brick surface and then to subtract one-half the thickness of the brick surface. This rule seems to apply also to the monolithic type of brick pavement.

Materials for the Concrete.—The requirements for the cement and fine aggregate for use in the concrete base are identical with those imposed for the concrete pavement. The coarse aggregate will not be subjected to direct wear and only such requirements as to cleanness and soundness should be imposed as will insure concrete of the desired strength. The aggregate can be somewhat softer than would be permissible for use in a wearing course, but satisfactory coarse aggregates will usually have a French Coefficient greater than 5. There is a tendency to use unscreened gravel and poorly graded broken stone more freely for base courses than for wearing course concrete. This is permissible if economical in a particular location. In any case the mixtures should be carefully designed to produce concrete of the flexural strength assumed in designing the concrete base.

Macadam Base Foundation.—In locations where sand and cement are not readily available and consequently are high-priced, the base is sometimes constructed of water-bound macadam. This type of base can be used successfully if the traffic does not include any considerable number of loads in excess of 4 tons. Eight inches is the minimum thickness for such a base and the construction must be of the best to insure a stable pavement. The base is constructed in exactly the same manner as a water-bound macadam wearing course.

Bituminous Macadam Foundation.—Bituminous macadam is used to a limited extent for the base course and when properly constructed is satisfactory. If it is economical in any locality there is no reason why it should not be used. It is usually at least 8 in. thick and is constructed of the materials and in the manner prescribed for standard bituminous macadam of the hot-mixed type.

Brick Foundation.—The earlier brick pavements were built with a base course of No. 2 paving brick but this type of construction is now obsolete. There may be an occasional situation where this type of construction would serve, but in general it is

inadequate for the traffic on any highway that requires a brick pavement.

CURBS

Brick pavements for streets are designed with combined concrete curb-and-gutter, straight concrete curbs, or stone curbs, preferred in the order given. Stone curbs are rather infrequently used but the choice between the other types is largely a matter of personal preference. Some economy results from the use of the concrete curb-and-gutter because the gutter slab costs less per unit of area than the brick pavement. Brick rural highways are usually designed with a marginal curb of concrete which is integral with the concrete base.

BEDDING COURSES FOR THE BRICK

The bedding course affords a means of adjusting the brick so that the upper surfaces will form a smooth pavement, and serves as a means of equalizing possible unevenness in the concrete foundation.

The bricks will not all be of exactly the same depth, and some will have slight imperfections such as kiln marks or slightly warped or checked surfaces. These defective surfaces are placed on the bedding course when the brick are laid. In the operation of laying, there will be some unevenness due to lack of skill on the part of workmen. When the brick surface is rolled, all of these inequalities are removed, the brick being pressed into the bedding course until the upper surfaces conform to the desired contour of the pavement.

The bedding course should be thick enough to permit rolling the brick without breaking them, and a rather extensive study indicates that 1 in. of bedding material is about the practical minimum. On the other hand, it seems to add an unnecessary expense if the bedding course is made thicker than 1¼ in.

The mortar bedding course does not tend to reduce the amount of impact transmitted to the concrete base, but it appears that the sand bedding course does absorb some of the impact and the same is probably true of the mastic and granulated slag bedding courses.

Sand Bedding Course.—The sand bedding course consists of about 1 in. of fairly clean, moderately fine sand. There is a tendency for the sand to shift under the brick, due to the vibration from traffic, and for it to sift into cracks in the concrete base, thus allowing the brick surface to settle unevenly. If

water enters along car tracks or cracks in the pavement, that has a tendency to cause the sand to shift. These defects of the sand bedding course are of no great significance where traffic is moderate but may become serious when the traffic is dense and the individual loads heavy.

Mortar Bedding Course.—The mortar bedding course was developed to secure a supporting layer that would not shift under the brick. The mortar is composed of 1 part cement, and anywhere from 4 to 6 parts of sand. These are dry-mixed and spread on the base in a layer about 1 in. thick. After the brick have been rolled, the pavement is sprinkled and thus water is provided for hydrating the cement which, upon setting, affords a bedding course that is especially desirable on streets with heavy traffic and where there are car tracks on the street.

Bituminous-mastic Bedding Course.—The bituminous-mastic bedding course consists of sand to which is added about 6 per cent by weight of light tar or asphaltic oil.[1]

The materials are heated and mixed, usually in a portable mixing plant, and then used in exactly the same way as the mortar bedding course and for like conditions.

Granulated Slag and Screenings for Bedding Course.—Screenings obtained in the preparation of commercial crushed stone have been used for the bedding course under the brick, but difficulty is experienced in rolling the brick to a smooth surface because the screenings layer will not yield under the brick. For that reason this type of bedding course has never been widely adopted. Granulated slab which is prepared for the purpose has been used for the bedding course and is satisfactory. It is used in lieu of a sand bedding course.

FILLERS FOR BRICK PAVEMENTS

The filler serves to bind the brick together so that they will not be disturbed by traffic; supports the upper edge of the brick so as to reduce the abrasion, and closes up the surface so water and street liquids cannot penetrate to the bedding course. Bituminous materials or cement grout may be used for the filler.

The Grout Filler.[2]—The filler is composed of one part each of clean, sharp, fine sand and portland cement. The mixture, not

[1] See Chap. XIV for a description of these materials.

[2] The cement grout filler is seldom used any more, but a description is included since it has not been wholly discarded. The bituminous filler is much preferred for most locations.

exceeding one sack of the cement, together with a like amount of sand, is placed in a box and mixed dry until the mass assumes an even and unbroken shade. Water is then added, forming a liquid mixture of the consistency of thin cream.

The work of filling should be carried forward until an advance of 15 to 20 yd. has been made when the same force and appliance are turned back and cover the space in like manner, except that the mixture shall be slightly thicker for the second coat.

To avoid the possibility of thickening at any point the surface ahead of the sweepers and ahead of the mixture should be gently

Courtesy of Mr. Will P. Blair.

FIG. 93.—Applying grout filler to brick pavement.

sprinkled using a sprinkling can the head of which is perforated with small holes.

After the joints are thus filled flush with the top of the brick and sufficient time for hardening has elapsed so that the coating of sand will not absorb any moisture from the cement mixture, ½ in. of sand is spread over the whole surface, and in case the work is subjected to a hot summer sun, the sand should be sprinkled lightly for 2 or 3 days. The street should be kept closed for 10 days in warm weather, and for a longer time if the weather is cool.

Recently, a successful mechanical mixer for the grout filler has been perfected which considerably simplifies the mixing

operation. The mixer is drawn along on the brick surface after the rolling and delivers the grout to the brick through a swinging chute. The area of pavement under the mixer should be kept covered with a canvas to catch the drippings from the mixer.

Bituminous Fillers.—When a bituminous filler is used instead of a cement grout filler, the pavement is completed in the same manner as when the grout filler is used, up to the point where the filler is applied. Bituminous fillers must be poured into the joints at a temperature sufficient to insure adhesion to the brick. This varies with different classes of materials, but usually is about 400°F. for asphalt fillers and about 250°F. for pitch fillers.

Some specifications provide that the joints shall be filled without covering the surface of the brick. The device which is used for pouring the filler under such a specification consists of a cone-shaped vessel having at the point a cast-iron tip with an opening about $\frac{1}{4}$ in. in diameter. The pouring can is drawn along the crack between the rows of brick, the point resting in the crack. The opening in the point is controlled by means of a valve with a handle conveniently arranged so that the flow of the bituminous material can be adjusted. As the vessel is drawn along the bituminous material is allowed to flow out into the joint in sufficient quantity to fill it. A helper replenishes the supply in the pouring can from time to time so that the pouring goes on continually.

Squeegee Method.—The high labor cost of filling the joints by means of the pouring can has led to the general adoption of the less expensive squeegee method, which has practically superseded all other methods of filling the joints of brick pavements.

In the squeegee method, the hot filler is poured onto the surface from pails and then worked into the joints by means of long-handled squeegees. It is impossible to scrape all of the filler from the surface of the pavement by this method, but on the contrary, there remains a thin layer of filler on the surface. This layer will remain intact for several years and serves as a very satisfactory carpet coat. After the filler has been applied the surface is covered with a light dressing of sand to prevent the filler from adhering to the wheels of vehicles.

Quantity of Filler Required.—The quantity of filler required to fill the joints will vary with the size of the joint, which will depend upon the setters, but the average 4-in. brick surface will require about 12 lb. of filler per square yard of surface if filled by

the pouring method, and about 16 lb. if filled by the squeegee method. The 3-in. surface will require approximately three-fourths as much.

Quality of Bituminous Filler.—The following specifications provide a suitable filler to use by the squeegee method:

ASPHALT FILLER

The asphalt shall be homogeneous, free from water, and shall not foam when heated to 200°C. (392°F.) It shall meet the following requirements:

Specific gravity at 25°C.	Not less than 0.980
Melting point	Not less than 80°C.
Penetration at 25°C., 100 g., 5 sec	30 to 45
Penetration at 0°C., 200 g., 60 sec	Not less than 20
Penetration at 46°C., 50 g., 5 sec	Not more than 100
Loss at 163°C., 5 hr	Not more than 1 per cent
Penetration of residue at 25°C., 100 g., 5 sec	Not less than 80 per cent of the original
Total bitumen (soluble in carbon disulphide)	Not less than 99 per cent
Organic matter insoluble in carbon disulphide	Not less than 0.7 per cent

Bituminous-mastic Filler.—The mastic employed for the joint filler consists of equal parts of a suitable bituminous material and fine sand. The materials are hot-mixed and the mixture is dumped on the brick surface and worked into the joints by means of squeegees.

This type of filler is especially adapted to locations where the pavement becomes very hot at certain seasons and where in consequence of steep grades the straight bituminous filler might flow from the joints. Any bituminous material that would be satisfactory for a carpet coat will serve for the mastic filler.

Expansion Joints.—Expansion joints are not required when the bituminous filler is used, but are generally employed for grout-filled brick pavements. The object of using the expansion joint is to take up the change in dimension of the grout-filled pavement following temperature changes. It is important on street pavements to provide for ample expansion along both curbs where the joints should not be less than ½ in. wide, for streets over 14 ft. wide, and not less than 1 in. for streets between 25 and 40 ft. wide.

Opinion differs as to the necessity of transverse joints. It seems to be well established that they may be omitted in regions where excessively high summer temperatures do not prevail, because expansion will not produce critical compression in the brick surface. Expansion crosswise of the pavement will move the curbs because they do not have sufficient support to withstand the pressure. Where summer temperatures are high, transverse expansion joints are used at intervals of between 50 and 100 ft.

Expansion joints are formed by a wooden strip set into the brick surface as the brick are laid. After the grout filler has hardened the wooden strips are removed and the joint filled with a straight asphalt filler or with a mastic.

There are several prepared joint fillers which consist of sheets of a bituminous material of the requisite width and thickness to fill the joints. This filler is set in place as the brick are laid.

BRICK RURAL HIGHWAYS

Design.—Brick rural highways are now called upon to carry loads that are the equal of those encountered in the cities, if the very few exceptional special loads that sometimes use city streets are disregarded, and the design must be based on heavy loads applied at the edge of the pavement. The principles of design that have been discussed in connection with city pavement are applicable to the rural highway if account is taken of the edge load to which the rural highway is subjected.

The concrete foundation is very generally employed and the marginal curbs, which are intended to retain the brick, are constructed integral with the concrete foundation. The marginal curbs are generally 6 in. wide and finished level with the brick surface, except on long grades where it is desired to retain the storm water on the pavement to reduce erosion. On such grades the cross-section is usually identical with that used for city streets except for the width. The bituminous filler is employed to fill the spaces between the brick and expansion joints are omitted.

The various operations involved in the construction are carried out very much as for city pavements, the requirements for materials are the same, and the necessity for careful workmanship equally imperative.

CONSTRUCTION METHODS

The construction operations in connection with brick pavements are organized with a view to securing maximum production with the plant available. The brick are hauled to the site and stacked outside the lines of the pavement as soon as the rough grading is completed. On city work the curb is next constructed followed in order by the construction of the concrete base, bedding course, brick course, and finally the usual cleaning up.

Earth Subgrade.—The earth foundation upon which the concrete foundation rests is known as the "subgrade or roadbed." After the pavement is completed, surface water will be cared for in pipe drains and will have little tendency to percolate to the subgrade. The pavement usually lies below the level of the parkings and lawns alongside, and underground water will be likely to percolate under the pavement base but the amount is generally insignificant. Porous soil and unfavorable topographical conditions may create a situation in which the subgrade will become water-saturated and unstable unless special precautions are taken to prevent it. The best precaution is to install suitable underground drainage. Sometimes storm-water sewers or sanitary sewers along the street will afford sufficient drainage, but in other instances additional tile drains must be laid. These are usually placed just back of the curb line and at a depth of 4 to 6 ft.

For a country brick road the drainage must be taken care of as faithfully as if the road were to have no hard surface and according to the principles discussed in Chap. III.

The subgrade is brought to the proper elevation and cross-section by excavating or filling as the case may be. If fills of less than 2 ft. are constructed they are built up in layers about 6 in. thick and each layer is rolled before the next is placed. For satisfactory results, the soil must be moist when it is rolled. Where cuts are made, it is advisable to plow within 2 or 3 in. of the subgrade and then roll to compress the subgrade to the proper elevation. The exact amount that the subgrade can be compressed by rolling varies with the type of soil and its condition, but it can readily be determined for each particular instance. A roller weighing 8 to 10 tons should be used for compacting the subgrade.

When finished, the surface of the subgrade should be true and smooth and so compact that the roller makes no perceptible

track upon it. Unevenness in the subgrade means inequalities in the thickness of the concrete base. Any high places in the subgrade may reduce the thickness of the base to such an extent as to reduce appreciably the load capacity of the pavement.

Placing Foundation Course.—When the concrete base is used it is constructed in a continuous operation for the full width of the pavement. For pavements up to 30 ft. wide some form of template can be used to shape the concrete to the proper form and thickness. For the wider pavements the concrete is shaped by means of shovels and lutes to elevations shown by pins driven in the subgrade. The concrete base should be of uniform texture, free of porous areas, and with no stones projecting out of the mortar.

Both the dry-batch and the wet-batch methods of construction are employed and the proportioning and mixing are rigidly controlled. Usually the minute mix is specified and the water limited to that which will produce a slump of 2 in. or less.

After the base concrete has been placed it is sprinkled daily for a week and the subsequent operations in connection with the construction should not begin for at least 10 days.

Placing the Bedding Course.—The bedding course material is spread on the base in a layer about ½ in. thicker than specified for the bedding course. Strips of wood of the thickness specified for the finished sand cushion are placed on the base near the curb and at the crown, and on top of these is placed a second set of strips ½ in. thick. The bedding course is struck off by dragging a screed along these strips, thus spreading the sand to the thickness of two strips combined. The screed is shaped to the proper cross-section and the cutting edge is shod with a metal plate. If the pavement does not exceed 25 ft. in width the screed may be constructed to span the entire width of the pavement, but for wider pavements it is better to span half the width.

After the bedding course has been struck off it is rolled thoroughly with a hand-operated roller about 30 in. long and weighing about 15 lb. per inch of length. The ½-in. wooden strips are then removed and the screed is drawn along the remaining strips and thus the well-packed sand is trimmed to exact thickness. The sand bedding course is most readily handled if it is in a moist condition, this being better than to have the sand either very wet or very dry.

Laying the Brick.—The brick are laid in straight rows at right angles to the center line except at intersections, where they are either laid parallel to the diagonals of the intersection or in the herringbone pattern. The end joints in each row are placed opposite the middle of the brick in the adjoining rows and part brick are used only in starting rows and making closures.

Lug brick are laid with the best face up and the lugs all in the same direction. After five or six rows have been placed they are closed up by driving with a maul. This is done to make the cross-joints as close as the lugs will permit. If lugless brick are used they are laid loosely and the courses are driven up just enough to keep them straight.

After the brick have been laid, the chips and spalls are swept off the surface, which is then inspected and all defective brick are replaced with good brick.

Rolling the Brick.—The brick are then rolled with a tandem roller weighing between 4 and 6 tons. The object of the rolling is to bring all the brick to a true surface, individual brick being adjusted by being pressed into the bedding course. If any are too low they are removed and a little material added to the bedding course. The rolling is started at the curb, the roller moving parallel to the curb and gradually working to the crown. The roller is then taken to the opposite side of the street and the operation repeated. After the first rolling, the surface is again inspected and all broken or spalled brick removed and replaced. The brick are again rolled, this time diagonally across the pavement, first in one direction and then in the other. Sometimes a final rolling parallel to the curbs is required. The brick cannot be rolled if the sand cushion is wet without forcing the sand up between the brick to an extent that will interfere with the penetration of the filler.

If the mortar bedding course has been used, the pavement is sprinkled after the rolling to furnish water for the cushion, provided the bituminous filler is to be used, but if the cement grout filler is to be used the sprinkling is done just before the grout is poured.

Monolithic Brick Pavements.[1]—The monolithic type of brick pavement is constructed by laying the brick-wearing surface

[1] Blackburn, W. T., "Brick Road Built Monolithic at Paris, Illinois," *Eng. Record*, Vol. 72, No. 2, p. 54, July 10, 1915.

directly on a freshly placed concrete base and rolling the brick to bed them in the concrete before it takes a set.

The general design and the requirements for materials are the same as for any other high-class brick pavement.[1] When constructed on a country road no marginal curb is used.

The subgrade is carefully prepared and thoroughly rolled and during the construction the vehicles used for hauling aggregates are kept on the side of the road as much as possible if the work is on a country road. The roller is kept at work on the subgrade, smoothing out the wheel tracks that are made by the trucks when they turn in on the subgrade to deliver concrete materials.

The concrete is mixed to a plastic consistency and struck off to the proper cross-section by means of a special templet which is a feature of the construction equipment that has already been developed for this particular type of pavement. It is shown in Fig. 92.

The templet consists of one I-beam crossbar and one channel crossbar, spaced about 2 ft. apart and supported on the steel side forms by means of rollers. The templet is drawn along the side forms by the mixer. The forward member of the templet cuts the concrete base $3/16$ in. below the finished grade, and dry mortar of one part cement and five parts sand is spread over the concrete ahead of the rear bar of the templet. This dry mortar is spread over the base by the rear part of the templet and the brick are laid directly thereon.

The setters place the brick carefully with the best side up and the lugs all in the same direction, and while the laying cannot proceed quite as rapidly as on the sand cushion, probably under favorable conditions a laborer can lay three-fourths as many per day as he would on a sand cushion.

The carriers who deliver the brick to the setters walk over the brick already laid and apparently do not disturb them, although as a precaution a 1 in. plank is laid over the brick where the carriers first step onto the newly laid surface. The bats are cut and set as fast as the courses are completed so that the surface is kept finished up close to the setters.

As fast as the laying is completed the surface is rolled with a hand roller about 30 in. in diameter and 30 in. long and having a weight of about 600 lb. The rolling irons out the little irreg-

[1] Specifications, Illinois Highway Department, 1928, p. 62.

ularities of the surface and beds the brick slightly in the dry mortar on the concrete base.

The surface is grouted in the manner described for the brick surface on a sand or mortar cushion.

It will benoted that the feature of this construction is that the brick are laid directly on the concrete base before the concrete hardens. A thin mortar bedding course is used merely to smooth the surface of the base.

CHAPTER XIV

BITUMINOUS ROAD AND PAVEMENT MATERIALS

Bituminous road and pavement materials are of two general types: asphalts and tars. Asphalt has been known and used from time immemorial, being found in the natural state in surface deposits in many parts of the world and called "asphalt" or "pitch" without regard to the actual composition or consistency of the material. The earlier applications were as cementing agents in building or conduit construction or as waterproof coatings for reservoirs, roofs, and buildings. It is only in comparatively recent times that the asphalts have been utilized extensively for road surface construction.

Tars are bituminous materials which are a by-product of the destructive distillation of coal in connection with the manufacture of coke and gas.

THE NATURE AND VALUE OF BITUMINOUS ROAD MATERIALS

Although asphalts and the tars resemble each other in general appearance and have some properties in common, they are actually very different in composition and in their behavior and usefulness for road construction. In discussing the general problem of bituminous road construction, it is necessary to keep clearly in mind the fact that there are important differences in the behavior of these two groups of bituminous materials.

Sources of Asphaltic Materials.—Asphaltic materials are found in natural deposits in many places, but the materials obtained from the Trinidad and the Venezuelan deposits are the ones most likely to be encountered by the road builder in the United States.

The discovery that the residue from the refining of petroleums of certain types was an asphalt that could be used for road construction has resulted in a very great expansion of the use of petroleum asphalts in road construction, and many varieties of the asphaltic materials now employed for highway purposes are obtained in the refining of petroleum.

Geological Origin of Asphalt.—Asphalts have their geological origin in the long process of decomposition of marine animal, or

vegetable, matter that resulted in the formation of the petroleum pools. Probably all of the native ("natural") asphalts once existed as ingredients of petroleum.

The petroleums vary in composition from those which consist largely of paraffinacious compounds to those which are almost wholly devoid of such substances. It has become the practice to refer to those first mentioned as "paraffin-base" petroleums and to those wholly devoid of hydrocarbons of the paraffin series as "asphalt-base" petroleums. Petroleums that contain some hydrocarbons of the paraffin series are known as "mixed-base" petroleums. The asphalt-base petroleums yield a residue that is adhesive and stable, with a characteristic appearance and feel which are readily recognized by those who have had considerable experience in handling these materials. The mixed-base petroleums yield a residue that has a greasy or unctuous feel which is readily detected by one accustomed to handling the materials. These paraffin ingredients serve as lubricants rather than binders and in addition tend to oxidize slowly upon exposure to the air, leaving a residue that is powdery or flaky and has no binding value.

Formation of Native Asphalt Deposits.—The native asphalts are residues formed by the weathering of petroleum that has reached the surface of the earth or cavities exposed to air and has undergone a long period of weathering during which some of the lighter constituents of the petroleum have evaporated. However, the deposits of native asphalts vary in consistency from fluid malthas (page 353) to materials that are hard enough to carry the weight of the excavating machinery used in mining the material. The native asphalts that are used for road purposes are exudations from pools of asphalt-base petroleums or mixed-base petroleums containing but little paraffin.

There are also found in many places exudations from mixed- or paraffin-base petroleums in the form of waxes and pitches which are employed extensively in the arts but have no particular significance to the road builder.

As the petroleum oozed through the porous layers of earth or through crevices in the earth's crust, it gathered up fine sand or clay which became intimately mixed with the asphalt, and this admixture of clay and sand characterizes each of the various deposits. In many cases it is possible to determine the origin of a sample of asphalt from the character of the inert material found

mixed with it. Some deposits of natural asphalt contain only a small percentage of material of this character, whereas in other cases the amount of inorganic matter may run to as high as 40 or 45 per cent by weight. Certain deposits of native asphalt are found in swampy tropical areas where organic matter from the adjacent growth has mixed with the asphalt. Many native asphalts contain some organic material insoluble in carbon disulphide, whereas others contain no considerable amount of such material. Here, again, is a means of identifying certain kinds of native asphalts.

Sources of Tars.—Tar is a by-product of the refining of the gas produced in coke ovens and is also produced in the manufacture of gas from bituminous coal or from fuel oil (the so-called "water-gas" process). Enormous quantities of this material are produced, and it is prepared for use as a road material by various processes that bring it to the proper consistencies and quality for use. The two varieties of tar are known as "coal tar" and "water-gas tar," respectively. Tar, as well as asphalt, in addition to its application to road construction, is extensively used for the preparation of roofing papers, insulating compounds, and various other purposes that utilize the waterproofing and insulating characteristics of this material.

Bituminous Cement.—The road builder is interested in bituminous materials because it is possible to produce from them a *bituminous cement* which is useful in binding together the particles of mineral aggregate that constitute the wearing surface of a road. For each type of highway construction a bituminous cement is required that will have not only durability and toughness but also the correct consistency for that type. That is, under the highest temperature to which it will be subjected by the natural climatic conditions in the region in which the road is built, the bituminous cement must be firm enough to be an effective binder for the mineral aggregates, and, conversely, it must not become brittle or lose its binding power at the lowest temperature to which the road surface is subjected. If the bituminous cement is incorporated in the mixture in a *very thin film* over particles of finely divided mineral aggregate, the consistency must be somewhat softer at the time of mixing than in the types of construction in which the bituminous binder is incorporated in open-textured mixtures where the film of the bituminous cement may be rather thick and where there may be

small deposits of the bituminous material in void space in the mixture of asphalt and mineral aggregate. For each type of bituminous road construction in an area there is a certain consistency of the bituminous cement necessary for stability in view of the nature of the mixture and the climatic conditions in the region. It is possible by well-understood processes to prepare bituminous cements of any desired consistency within the tolerances found to be admissible for materials of this character.

All the bituminous cements have the property of changing consistency with temperature, and in specifying the consistency of a bituminous cement there is always included a statement of the temperature at which the consistency is to be determined.

Consistency is measured in several ways, each adapted to a particular type of bituminous material.

1. The penetration test is employed for asphalt cements that are solid or semisolid at average summer temperatures.[1]
2. The float test is employed for asphaltic materials too soft for the penetration test and too hard for the viscosity test, and for tars, which cannot be tested by the penetration method because of the free carbon content.[2]
3. The viscosity test is employed for determining the consistency of bituminous materials that are too soft for either of the other standard tests.[3]

What has been said about the consistency of bituminous materials applies primarily to asphaltic products. However, tars are prepared as binders to be used for cold applications, and their consistency is measured by the viscosity method. Tars are not extensively used in types of construction employing the hot-mixing process. Semisolid tars are employed for penetration types of construction and for the fillers in expansion joints or in the spaces between the blocks of various types of block pavement. The consistency of a material of this type is determined by the

[1] The penetration test is made in accordance with the standard method adopted by the American Society for Testing Materials known as D5-25 and described in the 1936 Book of Standards, II, p. 1080.

[2] The float test is performed in accordance with the standard method adopted by the American Society for Testing Materials, known as D139-27 and described in the 1936 Book of Standards, II, p. 1061.

[3] The viscosity of emulsified asphalts is determined by the tentative standard method of the American Society for Testing Materials, known as D244-35T, described in the 1935 Book of Tentative Standards, p. 843. Other bituminous road materials are tested in accordance with A.S.T.M. Standard Method D88-33, described in the 1936 Book of Standards, II, p. 982.

float test or, if too hard for the float test, by the softening-point test, cube-in-water method.[1]

Incorporating Bituminous Cement in Mixtures.—A bituminous cement that is sufficiently firm to serve as an adequate binder for mineral aggregate at the temperatures existing in a road surface will be too viscous to mix with the mineral aggregate. That is, for most road purposes the bituminous cement must be semisolid or solid at air temperature to serve as an adequate binder.

Three methods are employed for temporarily bringing an asphalt cement to a condition that is sufficiently fluid to permit it to be mixed with the mineral aggregate and at the same time insure that within a reasonable time it will resume the consistency required for the stability of the road.

1. The oldest and perhaps most widely used of these methods of securing the fluid consistency necessary for mixing is to heat the bituminous material to a temperature of about 350 to 400°F. and to heat the mineral aggregate to about the same temperature and then mix the fluid hot bituminous material with the hot aggregate. When thorough mixing has been achieved, the bituminous mixture is spread and rolled before it cools sufficiently to become unworkable. In this process the bituminous material that is to be melted is either supplied by the dealer at a predetermined consistency or is fluxed to the desired consistency at the mixing plant. The hot-mixing method is widely used in the construction of sheet-asphalt, asphaltic concrete, and asphaltic macadam pavements.

2. The bituminous cement may be softened by dissolving it in kerosene, gasoline, or naphtha, or in mixtures of these materials. The materials softened in this way are called "cut-backs." By this process the bituminous cement can be brought to a fluid consistency so that it can be mixed with mineral aggregates while cold, or it may be softened with kerosene just enough to permit mixing at fairly low temperatures when the aggregates have been heated. It is also desirable to know in advance the period of time required for the bituminous cement in the mixtures produced in this way to reach its hardest consistency after the material has been incorporated in the road surface, and commercial cut-backs are designated as slow curing (S.C.), medium curing (M.C.), and rapid curing (R.C.) according to the time required for the solvent to evaporate.

3. The asphalts may be prepared in the form of emulsions with water to which has been added an emulsifier; these emulsions are fluid at air temperature and can be mixed with cold mineral aggregates by means of blade graders or portable mixers. The road surface reaches its maximum stability when the water has evaporated from the mixture, leaving the bituminous

[1] The softening-point (cube-in-water method) test is made in accordance with the standard method adopted by the American Society for Testing Materials, known as D61-24 and described in the 1936 Book of Standards, II, p. 989.

cement as a coating on the pieces of the mineral aggregate. So far as observations have gone up to the present time there is little or no change in the consistency of the asphalt cement in the mixture during the evaporation of the water from an emulsion.

Fluxing.—In the preparation of a bituminous cement to be used by the hot-mixing process, the hard asphalt is brought to the proper consistency by mixing with it a softer bituminous material which is called a "flux." The fluxing may be accomplished at the refinery where the petroleum asphalt is produced, but it is more frequently done at the mixing plant where the paving mixtures are being prepared.

Fluxes for asphaltic material are obtained from petroleum and are of two classes, semiasphaltic and asphaltic, the classification depending upon the predominating basic compound of which the flux is composed. Paraffin fluxes were at one time quite widely used but are not now available. A semiasphaltic flux is obtained principally from the mid-continent oil fields and is one of the best grades of flux obtainable at the present time. Asphaltic flux is obtained principally from the California petroleums. It is not a very soft material and consequently must be used in considerable quantities but is satisfactory when it is prepared so that it will remain soft in the pavement.

Certain properties are essential if the fluxes are to be satisfactory.

1. The flux should be prepared at high temperature yet without injury to the material from overheating. It is essential that the paving cement produced with the flux be stable over a long period of years; and unless the latter has been prepared at a temperature of about 450°F., it is likely to volatilize in service and leave a brittle pavement surface. Since the sheet-asphalt and asphaltic concrete pavement surfaces are usually mixed at temperatures ranging from 250 to 350°F., it is necessary to use a flux that will not catch fire, that is, "flash," at these temperatures, which is an additional reason why the flux should be manufactured at high temperature.

2. The flux should be fluid enough to have the property of mixing with hard bitumens in such a manner that it will reduce the harder material to a proper consistency without the necessity of using in excess of about 30 or 35 lb. of flux to 100 lb. of the harder bitumen. This characteristic is only partly dependent upon the fluidity of the flux, being influenced also by the character of the hydrocarbons of which the flux is composed.

3. The flux should consist of stable compounds so that it will not change after it has been incorporated in the pavement. This property should not be confused with volatility. A volatile flux will evaporate and cause the pavement to become brittle. Other fluxes are composed of

unstable compounds that change (perhaps oxidize) in a pavement with the passage of time. The result may be a bituminous cement that is of good binding properties when the pavement is laid but loses its binding properties with time.

Selection of Bituminous Material.—Commercial bituminous materials are most conveniently classified according to the purpose for which they are manufactured. For most kinds of bituminous construction there are large numbers of commercial products or brands of material available, representing many varieties and types.

The first consideration in selecting a material for a specific piece of construction is to secure one that is known to be satisfactory for that particular purpose. As has already been mentioned, this cannot be determined absolutely by means of laboratory tests that have for their purpose the determination of the inherent properties of the materials. Asphaltic materials are almost universally employed for the hot-mixed types of construction such as asphaltic concrete, sheet asphalt, and hot-mixed bituminous macadam. For these purposes oil asphalt from the mid-continent and California petroleums are widely used, and the native asphalts are also quite satisfactory and are used in large quantities. The asphalt employed for hot-mixed pavements may be purchased at the proper consistency; it may be prepared at the mixing plant by fluxing a hard asphalt; or a suitable slow-curing cut-back asphalt may be employed. For penetration macadam construction and for the various mixed-in-place and surface treatment types, both the tars and asphalts are extensively employed. In any of these cases, however, it is desirable to adhere to the rule that the bituminous material employed must have back of it a history of successful service. This means that when a new source of material is developed there must be some experimental construction in the promotion of the use of the material to gain experience with it and to establish its suitability for a particular purpose.

The second consideration is to secure a material of the proper consistency for the work. This involves two things, if the material is to be used by one of the cold-mixing processes. The first is that at ordinary air temperature the material delivered on the work must be sufficiently fluid to permit mixing with the mineral aggregate. The second is that after the bituminous material has cured by exposure to the air, the residue must be of

the proper consistency for the traffic and climatic conditions to which the road surface is exposed. Those asphalt cements which are to be used in a hot-mixing process must have the proper consistency when the finished road surface has reached air temperature, which will be several hours after the mixing has taken place. The use of cut-back asphalts in the hot-mixing process involves the still further consideration that when they have finally reached a stable condition through the evaporation of the lighter oils they will still be at the proper consistency for stability.

Standard Consistencies of Asphalts.—The asphalt cements that are to be used in a hot-mixing process and are not of the cut-back variety will change consistency very little in the mixing process, and the consistencies of these materials have been fairly well standardized. The United States Commercial Standard Consistencies for asphalts are given in Table XXIV, and specifications usually provide for a consistency complying with one of these standards. It will be apparent that the several penetration limits provided for in these standards make it possible to select a standard material that will be satisfactory for any climatic condition likely to be encountered in the United States.

TABLE XXIV.—NORMAL PENETRATION LIMITS FOR COMMERCIAL ASPHALT CEMENTS

77°F., 100 g., 5 sec.

25–30	50–60	85–100
30–40	60–70	100–120
40–50	70–85	120–150
		150–200

Asphalt Cements for Hot-mixed Types.—Native asphalt fluxed to the correct consistency and asphalts prepared from Mexican, mid-continent, or California petroleum are the principal types of asphalt cements in use for hot-mixed types of construction. Of the native asphalts, Trinidad and Bermudez are the only ones having extensive use in the United States. The Mexican oil asphalts came into favor with the discovery of the Mexican oil fields and have been widely used, but their production is gradually falling off. The mid-continent petroleums from certain areas are being used extensively for these types of construction. The California oil asphalts have been used successfully for many years.

Aside from these specification requirements intended to secure the desired type of material, the most important consideration for asphalt cement is to secure the proper consistency. This has already been mentioned repeatedly but is so important that it cannot be overemphasized. The manufacturers of oil asphalts prepare them at many consistencies. The native asphalts are fluxed at the mixing plants; it is a simple matter to secure any desired consistency for these materials; and that discourages standardization of consistencies for native asphalt cements. Table XXV was prepared by the Asphalt Institute[1] to serve as a guide in determining the consistency to adopt for any specific work and includes only commercially available consistencies.

TABLE XXV.—RECOMMENDED PENETRATIONS FOR ASPHALT CEMENTS FOR SPECIFIC TYPES

For surface treatment	150–350
For asphalt macadam:	
Warm climates	60–70, 70–85, 85–100
Northern climates	85–100, 100–120, 120–150
For asphaltic concrete (coarse graded)	40–50, 50–60, 60–70, 70–85
For stone-filled sheet asphalt and sheet asphalt	25–30, 30–40, 40–50, 50–60, 60–70, 70–85
For grout filler for stone block	50–60, 60–70, 85–100
For brick filler	25–40

Cut-backs and Emulsions.—In Table XXVI are given the principal properties of certain classes of bituminous road materials available on the market. It will be noted that these are grouped into three classes: those which are S.C. (see page 340), those that are M.C., and those that are R.C. The footnote to the table indicates in a general way the classes of construction for which each of the groups of materials is most commonly employed (see page 359).

OCCURRENCE AND PRODUCTION OF ASPHALT AND TAR

The term "asphalt" was first applied to materials that have exuded from the earth in the form of a viscous liquid which collected in pools or "lakes" and may or may not have hardened from exposure, and which might be fairly free from organic and inorganic matter that is not bitumen, or which might have impregnated porous rock or sand. The use of this term gradually expanded and is now applied to a wide variety of substances that

[1] HUBBARD, PRÉVOST, and BERNARD E. GRAY, "Asphalt Pocket Reference for Highway Engineers," p. 43, The Asphalt Institute, New York, N.Y., 1937.

are somewhat alike in physical characteristics. Some of these are obtained from natural deposits; some are produced in connection with the refining of petroleum.

Oil Asphalts.—Oil asphalt is the residue from the fractional distillation of petroleum and is therefore somewhat analogous to the residue that remains after exudations of petroleum have weathered from exposure to the elements. In either instance the character of the residue will be determined largely by the nature of the petroleum from which it came. Oil asphalts are classed as asphalt-base, mixed-base, or paraffin-base residue; but only the two first named have a general usefulness in highway construction. These residues are produced in a variety of consistencies and are of varying degrees of suitability, depending upon the physical characteristics of the particular material.

The asphalt-base residues and those mixed-base residues which contain but little paraffinacious material are most widely used for pavement surfaces. In North America they are obtained principally from the petroleums of California and especially from those of Mexico. Various petroleums obtained in the mid-continent fields of the United States yield residues that can be used for some kinds of highway construction other than the sheet types of pavements and bituminous-road surfaces.

NATURAL ASPHALTS

Trinidad Asphalt.—The largest and best known of the native asphalt deposits is that on the island of Trinidad in the British West Indies, which is known as the Pitch Lake. This deposit is in the nature of a lake of about 125 acres extent, having a depth of more than 135 ft. in places. The material obtained from this deposit contains pieces of wood, gas, water, and a considerable quantity of fine sand and clay. The refined material is of uniform quality and contains 56.5 per cent of pure bitumen, the remainder being sand, clay, and organic matter insoluble in carbon disulphide. Other deposits of similar material are scattered over the islands, but they are of much less importance commercially. Trinidad asphalt has been long and widely used as a binder for sheet-asphalt and asphaltic concrete pavements. It is unlike any other asphalt known.

Trinidad Petroleum.—A heavy asphaltic petroleum oil is obtained from wells on Trinidad Island, and this oil field has been extensively developed. This material contains a relatively

small amount of the lighter oily constituents common to petroleum, and the body of the oil is of a truly asphaltic nature and exceedingly sticky and stable. This material is soluble in carbon disulphide to the extent of 99.9 per cent. It is refined and marketed as a dust layer and as a binder for the construction of bituminous carpets.

Bermudez Asphalt.—Another well-known deposit of native asphalt is on the north coast of Venezuela and is known as the *Bermudez deposit.* It is a comparatively shallow deposit, being in most places about 7 ft. thick, but has an area of about 900 acres. The material has exuded from the earth and spread over a swampy area which is often covered with water. It contains decayed vegetable matter, sticks, and a little clay and water. The water-free material is somewhat variable in composition and contains from 93 to 97 per cent pure bitumen with an average of 95 per cent. The remainder is clay and organic matter insoluble in carbon disulphide. Bermudez asphalt is widely used as a binder for sheet-asphalt, asphaltic concrete, and macadam surfaces.

Cuban Asphalt.—Several deposits of native asphalt exist in Cuba, most of them of rather recent development. In general, the Cuban asphalt that has been utilized for paving purposes resembles slightly the Trinidad Lake asphalt but is more variable in its characteristics. It contains from 65 to 75 per cent of bitumen, the remainder being sand and clay and organic matter insoluble in carbon disulphide. Cuban asphalt is used to some extent for sheet and asphaltic concrete pavements and for macadam surfaces.

Mexican Asphalt.—Deposits of native asphalt also exist along the Gulf Coast of Mexico and for a short distance inland in the Tuxpam district. These deposits are variable in composition but are being developed rapidly and will probably be used extensively in the future. The various deposits contain from 60 to 99 per cent pure bitumen. The Mexican asphalts have been used principally for sheet and asphaltic concrete surfaces.

Miscellaneous other deposits of native asphalt are found throughout the world, and the material has been an article of commerce from the earliest times. Asphalt is found in the vicinity of the Dead Sea, and its properties and value have been known locally from the earliest Bible times. The Dead Sea deposits have not been developed on a large scale. Other deposits in Asia and Europe have been known and utilized for

centuries, and many European cities have been paved with asphalt obtained from these sources.

Deposits of minor importance are known in South America and in Texas and California.

Natural Rock Asphalt.—In many parts of the world there are extensive deposits of limestone or sandstone rock which by some process of nature have become impregnated with asphalt, usually in the form of a soft maltha. The best known deposits in the United States are in Kentucky, Oklahoma, Texas, and Southern California; those in Germany, France, Sicily, and Russia are best known abroad. This class of material has had a limited use for sheet-asphalt surfaces in the United States but has been extensively used in Europe. As might be expected, the rock asphalts vary greatly in character, and few of them can be used for paving purposes without the addition of either sand or bitumen or both. The rock asphalts are also extensively used for mastic floors in factories and laboratories.

Refining Native Asphalts.—It is doubtless apparent from the foregoing that the various native asphalts differ greatly in consistency and other physical characteristics and that all must be prepared in some manner to render them suitable for paving purposes. In some of the deposits the materials vary from hard pitches to soft asphalts or malthas (page 353) which flow slowly at ordinary temperature; in others the consistency is fairly uniform.

A brief description of the method of refining the Trinidad Lake asphalt will indicate in a general way the usual preparation necessary before natural asphalts can be used for paving purposes.

The Trinidad Pitch Lake is some miles inland and about 140 ft. above sea level. The asphalt is broken out of the surface in irregular pieces by means of picks and is loaded on to small cars. These are hauled out from the region of the lake, and the removable bodies of the cars attached to a cableway and transported to the dock where the material is loaded into vessels. When the cargo reaches the refinery (Maurer, N.J.) the asphalt is picked loose in the hold of the vessel and removed to the melting tanks. It is then heated by means of steam coils to a temperature exceeding 212°F., after which it is agitated by means of jets of steam from pipes in the bottom of the tank. As the heating progresses, the entrained water and gas are given off, and the sticks and other vegetable matter are removed. The material is then drawn off into barrels to be shipped to the place of use.

There it is fluxed with a suitable petroleum oil to reduce its consistency to that desired for the paving cement.

TARS

Coal tar is a by-product of the destructive distillation of coal. If the tar is produced during the manufacture of illuminating gas from coal, it is commonly called "gas-house" tar. If it is produced as a by-product in the manufacture of coke, it is known as "coke-oven" tar. The gas-house tar is produced at a higher temperature than the coke-oven tar and usually contains more free carbon (soot) than the coke-oven tar, owing to the differences in processes of which the tars are a by-product. Tars are almost pure bitumen, aside from the free carbon. They have the same general appearance as asphalts but have the characteristic and familiar tar odor which differs from the odor of any of the asphaltic materials. Tars are refined to prepare them for paving purposes, the process consisting in distilling off the water and lighter or more volatile oils, leaving a residue of the desired consistency. Coal tar is used for penetration and mixed macadam and for carpeting.

Cut-back Pitches.—Tars are used widely in the arts in obtaining the basic materials for dyes, medicines, etc.; and incident to manufacturing these products, residues are often obtained that are too hard for paving purposes. These are softened, or "fluxed," by means of lighter distillates obtained from tar as a by-product of manufacturing processes, and the resulting mixture is known as a "cut-back pitch" or simply as a "cut-back." These tars are manufactured with various consistencies suitable for the different classes of macadam roadwork.

Water-gas Tar.—Water-gas tar is produced as a by-product of the manufacture of illuminating gas from oil and water. It is the result of the destructive distillation or "cracking" of a petroleum oil. It has the same general appearance as coal tar and has much the same odor but is generally thought to be less useful for road purposes. It also differs in that it carries a much lower amount of free carbon than do the other classes of tars. It is refined and marketed for all classes of macadam construction.

PETROLEUM ASPHALT

Production of Petroleum Asphalt.—Petroleum asphalt, as has already been explained, is the residue from the fractional

distillation of petroleum after the lower boiling point products have been distilled off. In general, the constituents of petroleum may be grouped in four categories: (*a*) volatile oils such as naphtha and gasoline, (*b*) oils that volatilize slowly such as kerosene, (*c*) oils that are not volatile such as the lubricants, and (*d*) hard asphalt. Commercial products of petroleum origin that are of interest to the roadbuilder are the following: (1) crude petroleum, which obviously is a mixture of the four groups of products mentioned previously as *a*, *b*, *c*, and *d*; (2) residual asphalt oils which are mixtures of *b*, *c*, and *d* (these are also classed as fuel oils); (3) asphalt cement, which is a mixture of *c* and *d*; and (4) hard asphalt (*d*). Of these materials the residual asphalt oils and the asphalt cements are used directly in road construction, and the hard asphalt is the basis for the production of asphalt cement. Certain grades of crude petroleum are used as dust layers.

The residues from the refining of petroleum are not all suitable for the production of asphalt cements or cut-backs; on the contrary, the petroleum asphalt produced for road purposes is prepared from selected asphalt-base crude petroleums or mixed-base petroleums which yield a residue of proved merit. In some refineries the crude is refined primarily to secure the asphalt, as the other products contained in the crude are so limited in quantity that it scarcely pays to refine the oil. This has been especially true of certain Mexican crude petroleums.

Mixtures.—Many road binders and paving cements are mixtures of two or more of the classes or types of bituminous materials that have been mentioned. As an illustration, the practice of mixing California with Texas residues might be mentioned. Water-gas tars and asphaltic materials are also mixed in manufacturing some brands of road materials. These products often appear to have desirable physical properties, but their behavior under traffic has not yet been fully established.

Bitumen.—In order to secure suitable mixtures of mineral aggregate and bituminous cement the engineer must know the percentage of bitumen in a given bituminous material, because it is only the bitumen that has binding properties.

The native asphalts and the tars contain in addition to the useful binding agent, which is called *bitumen*, certain other substances that are inert in character and do not affect the behavior of the bitumen but must be taken into account in

determining how much bitumen there is in a given sample of the bituminous material. The native asphalts contain in addition to bitumen a small amount of fine sand and earthy material and sometimes small quantities of organic matter, which is the residue from decayed vegetable matter. The tars contain free carbon which is merely soot that becomes incorporated with the tar in the coking process.

THE TECHNOLOGY OF BITUMINOUS MATERIALS

The technology of bituminous materials deals with manufacturing processes, specifications, methods of testing and analysis, the effects of exposure to air on the quality of these materials, nomenclature, and similar factors. Certain phases of the technology of bituminous materials are of especial interest to highway engineers and will be presented from the viewpoint of their relation to the intelligent utilization of these materials in highway construction.

Definitions.—The first step toward an understanding of the technology of bituminous materials is to acquire a knowledge of the special terminology involved. The following definitions cover the more commonly used terms.[1]

1. *Asphaltenes.* The components of the bitumen in petroleums, petroleum products, malthas, asphalt cements, and solid native bitumens, which are soluble in carbon disulfide but insoluble in paraffin naphthas.

2. *Asphalts.* Black to dark-brown solid or semisolid cementitious materials which gradually liquefy when heated, in which the predominating constituents are bitumens all of which occur in the solid or semisolid form in nature or are obtained by refining petroleum or which are combinations of the bitumens mentioned with each other or with petroleum or derivatives thereof.

3. *Asphalt cement.* A fluxed or unfluxed asphalt specially prepared as to quality and consistency for direct use in the manufacture of bituminous pavements and having a penetration at 25°C. (77°F.) of between 5 and 250, under a load of 100 g. applied for 5 sec.

4. *Bitumens.* Mixtures of hydrocarbons of natural or pyrogenous origin or combinations of both frequently accompanied by their non-metallic derivatives, which may be gaseous, liquid, semisolid, or solid, and which are completely soluble in carbon disulfide.

5. *Bituminous.* Containing bitumen or constituting the source of bitumen.

6. *Bituminous emulsion.* A liquid mixture in which minute globules of bitumen are held in suspension in water or a watery solution.

[1] *Standard Definitions of Terms Relating to Materials for Roads and Pavements*, Serial Designation, D8-33, American Society for Testing Materials, 1936 Book of Standards, II, p. 1116.

7. *Blown petroleums.* Semisolid or solid products produced primarily by the action of air upon liquid native bitumens which are heated during the blowing process. (Also called oxidized asphalt.)

8. *Carbenes.* The components of the bitumen in petroleums, petroleum products, malthas, asphalt cements, and solid native bitumens, which are soluble in carbon disulfide but insoluble in carbon tetrachloride.

9. *Coal tar.* Tar produced by the destructive distillation of bituminous coal.

10. *Coke-oven tar.* Coal tar produced in by-product coke ovens in the manufacture of coke from bituminous coal.

11. *Consistency.* The degree of solidity or fluidity of bituminous materials.

12. *Cut-back products.* Petroleum or tar residuums that have been fluxed with distillates.

13. *Dead oils.* Oils with a density greater than water which are distilled from tars.

14. *Dehydrated tars.* Tars from which all water has been removed.

15. *Fixed carbon.* The organic matter of the residual coke obtained upon burning hydrocarbon products in a covered vessel in the absence of free oxygen.

16. *Fluxes.* Bituminous materials, generally liquid, in which the predominating constituent is bitumen, used in combination with asphalts for the purpose of softening the latter.

17. *Free carbon in tars.* Organic matter that is insoluble in carbon disulfide.

18. *Gas-house coal tar.* Coal tar produced in gas-house retorts in the manufacture of illuminating gas from bituminous coal.

19. *Liquid asphalt.* This is a trade term not subject to definition.

20. *Liquid bituminous materials.* Those having a penetration at 25°C. (77°F.), under a load of 50 g. applied for 1 sec., of more than 350.

21. *Native asphalt.* Asphalt occurring as such in nature.

22. *Oil-gas tars.* Tars produced by cracking oil vapors at high temperatures in the manufacture of oil gas.

23. *Penetration.* The consistency of a bituminous material expressed as the distance that a standard needle vertically penetrates a sample of the material under known conditions of loading, time, and temperature. Where the conditions of test are not specifically mentioned, the load, time, and temperature are understood to be 100 g., 5 sec., and 25°C. (77°F.), respectively, and the units of penetration to indicate hundredths of a centimeter.

24. *Petroleum.* Liquid bitumen occurring as such in nature.

25. *Pitches.* Black or dark-brown solid cementitious residues which gradually liquefy when heated and which are produced by the partial evaporation or fractional distillation of tars.

26. *Refined tar.* Tar freed from water by evaporation or distillation which is continued until the residue is of desired consistency; or a product produced by fluxing tar residuum with tar distillate.

27. *Rock asphalt.* Sandstone or limestone naturally impregnated with asphalt.

28. *Semisolid bituminous materials.* Those having a penetration at 25°C. (77°F.), under a load of 100 g. applied for 5 sec., of more than 10, and a penetration at 25°C. (77°F.), under a load of 50 g. applied for 1 sec., of not more than 350.

29. *Solid bituminous materials.* Those having a penetration at 25°C. (77°F.), under a load of 100 g. applied for 5 sec., of not more than 10.

30. *Straight-run pitch.* A pitch run to the consistency desired, in the initial process of distillation, without subsequent fluxing.

31. *Tars.* Black to dark-brown bituminous condensates which yield substantial quantities of pitch when partially evaporated or fractionally distilled and which are produced by destructive distillation of organic material, such as coal, oil, lignite, peat, and wood.

32. *Topped petroleum.* Petroleum deprived of its more volatile constituents.

33. *Viscosity.* The measure of the resistance to flow of a bituminous material, usually stated as the time of flow of a given amount of the material through a given orifice.

34. *Water-gas tars.* Tars produced by cracking oil vapors at high temperatures in the manufacture of carbureted water-gas.

The following definitions cover terms not included in the A.S.T.M. list but frequently employed in bituminous construction and in this treatise.[1]

35. *Asphalt filler.* An asphaltic product used for filling cracks and joints in pavement structures.

36. *Asphalt primer.* A liquid asphaltic road material of low viscosity which upon application to a non-bituminous surface is completely absorbed. Its purpose is to waterproof the existing surface and prepare it to serve as a base for the construction of a bituminous carpet or surface course.

37. *Asphaltic residual oil.* A liquid residue produced in petroleum refining, which contains little or no readily volatile constituents but yields asphalt upon evaporation at high temperatures.

38. *Cut-back asphalt.* Asphalt cement that has been rendered liquid by fluxing it with a light volatile petroleum distillate. Upon exposure to atmospheric conditions the volatile distillate evaporates, leaving the asphalt cement behind.

39. *Emulsified asphalt.* An emulsion of asphalt cement and water containing a small amount of emulsifying agent. (Emulsions are heterogeneous systems containing two normally immiscible liquid phases, one of which is dispersed as fine droplets or globules in the other.)

40. *Flux or flux oil.* A thick, viscous, non-volatile oil recovered from petroleum by distilling off the light volatile products present in the crude petroleum. It is used to soften hard asphalts, to any desired consistency.

41. *Gilsonite.* A hard, brittle, *native asphalt* occurring in various localities in rock crevices or veins from which it is mined like coal.

[1] HUBBARD and GRAY, *op. cit.*, p. 1?.

42. *Hard asphalt.* Solid asphalt that has a normal penetration of less than 10. To make it suitable for ordinary use it must be softened to desired consistency by combining it with flux oil.

43. *Lake asphalt.* A native asphalt occurring as surface deposits in natural depressions of the earth's crust.

44. *Maltha.* A very viscous asphaltic petroleum which usually hardens rapidly upon atmospheric exposure owing to volatilization of its lighter constituents.

45. *Mineral-filled asphalt.* Asphalt cement containing an appreciable percentage (usually between 10 and 50 per cent by weight) of very finely divided mineral matter passing the 200-mesh sieve.

46. *Petroleum asphalt.* Asphalt refined directly from petroleum. Most petroleum asphalt used in highway work is produced by merely distilling off the gasoline, kerosene, and other oils that hold it in solution. Asphalt is one of the constituents of asphaltic petroleums, which are refined primarily for its recovery.

47. *Powdered asphalt.* Hard asphalt crushed or ground to a fine state of subdivision.

48. *Preformed asphalt joint fillers.* Premoulded strips of asphalt cement, mixed with fine material substances, fibrous materials, cork, sawdust, etc. They are manufactured in dimensions suitable for insertion in construction joints.

49. *Refined asphalt.* Any asphalt that has been subjected to a refining process. In paving work this term, however, is usually restricted to asphalt that, after refining, is too hard for use in a given type of construction and must be softened to suitable consistency by combining it with *flux oil.*

50. *Steam-refined asphalt.* Asphalt that has been refined in the presence of steam during the distillation process.

Quality Tests for Bituminous Materials.—There is no basis as yet for correlating the inherent properties of bituminous materials with the quality of the material as a cementing agent. For some unaccountable reason there have not been developed through research in this important field adequate data to indicate the relations between laboratory tests and service behavior, except for consistency. As a consequence, a bituminous product must pass through the costly process of trial in road construction to establish its merit. When it has proved itself, subsequent specifications and tests are designed to insure that identical materials are furnished, except that the requirements as to consistency may be changed from time to time to meet the particular requirements of specific projects.

As an outgrowth of the situation explained in the foregoing paragraphs, it is found that specifications for bituminous materials often include provisions that have no direct relation to quality but, on the contrary, are intended to limit the materials

acceptable under their specifications to certain products that have proved satisfactory in service. Most specifications for bituminous materials have to be written on that basis in the present state of knowledge of the relation between the physical properties of a material and their suitability as road materials.

The situation with respect to writing specifications for tar is identical with that encountered in dealing with asphalt. There is no wholly dependable basis for correlating the physical properties of tar with its service behavior as a highway material, except perhaps as regards consistency. The quantity of tar produced in the United States is greatly in excess of the demand for any purpose, and certain manufacturers have developed a line of products for use in road construction. Through the method of trial and error they have finally hit upon formulas that produce materials acceptable for certain classes of construction.

Typical Specifications.—The following specifications for bituminous materials for various types of construction will serve to illustrate the practice discussed above.

A. Specifications for Bituminous Materials in the Standard Specifications for Construction Work for the Iowa Highway Commission

Tar.—Tar for the repair of concrete pavement and for filling joints and cracks except poured expansion joints, in concrete pavement, shall be homogeneous and free from water and shall conform to the following detailed requirements:

	Detailed requirements	Minimum	Maximum
1	Specific gravity	1.200	1.260
2	Float test at 50°C. (122°F.), seconds	90	120
3	Total distillate by weight, per cent:		
	To 170°C. (338°F.)	......	1.0
	To 270°C. (518°F.)	......	12.0
	To 300°C. (572°F.)	......	20.0
4	Softening point of residue (ring-and-ball method), degrees centigrade	......	60.0
5	Specific gravity of distillate	1.03	
6	Total bitumen, per cent	78	92
7	Inorganic matter (ash), per cent	......	0.5

Measurement shall be based on the volume of material at temperature of 60°F.

Asphalt.—Asphalt for the repair of concrete pavement and for filling joints and cracks except poured expansion joints, in concrete pavement, shall be homogeneous and free from water and shall conform to the following detailed requirements:

	Detailed requirements	Fluxed natural asphalt		Petroleum asphalt	
		Minimum	Maximum	Minimum	Maximum
1	Specific gravity at 77°F.	1.15	1.24	1.00	1.10
2	Softening point, degrees Fahrenheit	104	118	100	120
3	Penetration at 77°F., 100 g., 5 sec.	85	100	100	120
4	Penetration at 32°F., 200 g., 1 min.	20	......	35	
5	Loss on heating 50 g., 5 hr., 325°F., per cent	......	2.0	......	0.5
6	Flash point, open cup	350	......	450	
7	Inorganic matter naturally present, per cent	15	23	......	0.5
8	Total bitumen, per cent	70	80	99.5	
9	Bitumen soluble in carbon tetrachloride, per cent	99	......	99.0	
10	Ductility at 77°F., centimeters	50	......	50	
11	Penetration of residue from No. 5 at 77°F., 100 g., 5 sec., per cent of original	60	......	70	

B. SPECIFICATIONS FOR ASPHALT CEMENT FROM THE STANDARD SPECIFICATIONS OF THE DIVISION OF HIGHWAYS OF THE STATE OF CALIFORNIA

General.—The asphaltic cement used under these specifications shall be prepared from a California crude asphaltic petroleum. It shall be an oil asphalt of the required degree of penetration or a mixture of a refined liquid asphalt with a refined solid oil asphalt and must be free from admixture with any residues obtained by the artificial distillation of coal, coal tar, or paraffin oil. It shall be homogeneous and free from water.

Grade D.—Asphalt Grade *D* shall be an asphaltic product as described above and shall conform to the following requirements:

When a sample is heated for 5 hr. at a uniformly maintained temperature of 325°F., it shall not lose more than 2 per cent in weight; and the penetration at 77°F. during 5 sec. with a 100-g. weight, after such heating, shall not be less than 60 per cent of the original penetration.

The asphaltic cement shall be soluble in cold carbon bisulphide to the extent of not less than 99.5 per cent and in cold carbon tetrachloride to the extent of not less than 98 per cent.

The ductility at 77°F. shall not be less than 30 cm.

C. Specifications for Asphalt Cement from the Standard Specifications for Road Construction of the Minnesota Department of Highways

	Kind of asphalt		
	Oil	Bermudez A.C.	Trinidad A.C.
Specific gravity 25°/°C.	1.00+	1.05–1.07	1.20–1.25
Penetration at 25°C., 100 g., 5 sec:			
Asphaltic concrete	50–60	50–60	50–60
Sheet asphalt	40–50	40–50	40–50
Solubility in CS_2 (per cent)	99.5+	94.0+	60.0+
Ductility at 25°C., cm.	50.0+	30.0+	30.0+
Flash point (°C.)	175+	175+	175+
Loss at 163°C. 5 hr. not more than (per cent)	1	3	3
Penetration of residue at 25°C., 100 g., 5 sec., as compared to penetration before heating, not less than (per cent)	60	50	50

The following materials are recommended by the U.S. Bureau of Public Roads for bituminous treatments on sand-clay surfaces.[1]

Bituminous Materials.—The bituminous materials shall meet the following requirements for the purposes indicated.

D. Tar for First (Prime) Application

The tar shall conform to the following requirements:

1. Water, not more than 2 per cent.
2. Specific viscosity, Engler, 50 c. at 40°C. (104°F.), 8 to 13.
3. Distillation test on water-free material:
   - Total distillate, by weight, 0 to 170°C. (32 to 338°F.), not more than 7 per cent.
   - Total distillate, by weight, 0 to 235°C. (32 to 455°F.), not more than 20 per cent.
   - Total distillate, by weight, 0 to 270°C. (32 to 518°F.), not more than 30 per cent.
   - Total distillate, by weight, 0 to 300°C. (32 to 572°F.), not more than 35 per cent.
4. Specific gravity at 25°/25°C. (77°/77°F.) of total distillate to 300°C. (572°F.), not less than 1.01.
5. Softening point (ring-and-ball method) of residue from distillation test, not more than 60°C. (140°F.).
6. Total bitumen (soluble in carbon disulphide), 88 to 97 per cent.

[1] "Bituminous Surface Treatments of Sand-clay and Topsoil Roads," *Public Roads*, Vol. 10, No. 10, p. 207, January, 1930.

E. Asphalt for Second (Hot) Application

The asphalt shall be homogeneous, shall be free from water, and shall not foam when heated to 175°C. (347°F.). It shall meet the following requirements:

1. Specific gravity 25°/25°C. (77°/77°F.), not less than 1.000.
2. Flash point, not less than 175°C. (347°F.).
3. Penetration at 25°C. (77°F.), 100 g., 5 sec., 150 to 200.
4. Loss at 163°C. (325°F.), 50 g., 5 hr., not more than 2 per cent.
   a. Penetration of residue at 25°C. (77°F.), 100 g., 5 sec., as compared to penetration before heating, not less than 60 per cent.
5. Bitumen (soluble in carbon disulphide), not less than 99.5 per cent.
   a. Organic matter insoluble, not more than 0.2 per cent.

F. Cut-back Asphalt for Third (Seal) Application

The cut-back asphalt shall conform to the following requirements:

1. Specific gravity (at 25°C.), not less than 0.92.
2. Specific viscosity (Engler), at 50°C., 15 to 25.
4. Loss by evaporation, 50 g., 5 hr. at 163°C., not less than 23 per cent.
5. Solubility in CCl_4, not less than 99.8 per cent.
6. Distillation:
   Total distillate at 100°C., not more than 5 per cent (by volume).
   Total distillate at 150°C., not more than 15 per cent (by volume).
   Total distillate at 205°C., not more than 35 per cent (by volume).
   Asphalt content at 100 penetration, 63 to 75 per cent.

The specifications in Table XXVI cover the requirements for liquid asphaltic road materials to be used for various purposes as indicated in the footnotes to the table. These products are commercially available, and they are quite generally referred to in highway literature by their designation as S.C.1 or R.C.3, etc. These designations are quite understandable among those who use materials of this character.

G. Tars for Surface Treatments

1. (cold application) and second seal coat penetration macadam

The refined tar shall be homogeneous and meet the following requirements:

Water by weight, not more than (per cent)	2.0
Specific viscosity, Engler 50 cc. at 40°C	8–15
Total bitumen (soluble in carbon disulphide) not less than (per cent).	88
Distillation test on water-free material:	
Total distillate by weight, not more than	
0 to 170°C. (per cent)	7
0 to 270°C. (per cent)	32
0 to 300°C. (per cent)	42
Softening point of residue, not more than (degrees centigrade)	60

2. (HOT APPLICATION)

The refined tar shall be homogeneous, free from water, and meet the following requirements:

Float test at 32°C. (seconds)	60–150
Total bitumen (soluble in carbon disulphide) not less than (per cent).	80
Distillation test:	
Total distillate by weight, not more than	
0 to 170°C. (per cent)	1
0 to 270°C. (per cent)	17
0 to 300°C. (per cent)	25
Softening point of residue, not more than (degrees centigrade)	65

3. TAR, COLD-MIX TYPE) BRIDGE FLOORS AND PAVEMENT MAINTENANCE

The tar shall be homogeneous and meet the following requirements:

Water, by weight, not more than (per cent)	2
Specific viscosity, Engler 50 cc. at 40°C	35–80
Total bitumen (soluble in carbon disulphide) not less than (per cent)	80
Distillation test on water-free material:	
Total distillate by weight:	
0 to 170°C. (per cent	2–10
0 to 235°C. (per cent)	10–20
0 to 300°C. (per cent)	18–32
Softening point of residue, not more than (degrees centigrade)	70

Routine Tests of Bituminous Materials.—Certain routine tests are required in the control of quality and quantity of the bituminous materials employed in highway work. These tests are made in part to identify the materials, in part to insure quality, and in part to provide the data needed for determining the quantities actually used on a job or in a mixture. The tests most frequently used are the following:

1. *Sampling Bituminous Materials,* A.S.T.M. D140-25, 1936 B.S., II, p. 1088.[1]—A laboratory test on a sample of a material is valid, within the allowable tolerances, for the material actually subjected to the test. Whether or not it is valid for the supply from which the sample was taken depends wholly upon the adequacy of the method of sampling. If the sample is truly representative of the supply from which it was taken, then the test may legitimately be employed in judging the value of the material or the extent to which it complies with the purchase specifications.
2. *Penetration Test.*—Discussed on page 339.
3. *Viscosity Test.*—Discussed on page 339.
4. *Float Test.*—Discussed on page 339.

[1] This form of reference will be used throughout this section and should be read: American Society for Testing Materials Standard Method, Serial Designation, D140-25, found in 1936 Book of Standards. Part II, p. 1088.

TABLE XXVI.—PROPOSED SPECIFICATION REQUIREMENTS FOR LIQUID ASPHALTIC ROAD MATERIALS[1]

Test requirements	Material type												
	S.C. 1	S.C. 2	S.C. 3	S.C. 4	M.C. 1	M.C. 2	M.C. 3	M.C. 4	M.C. 5	R.C. 1	R.C. 2	R.C. 3	R.C. 4
Water and sediment, %	2.0−	2.0−	2.0−	2.0−									
Flash point, °F	150+	200+	200+	250+		150+	150+	150+	150+	80+	80+	80+	80+
Viscosity:													
Saybolt Furol at 77°F., sec	20–150				40–150								
Saybolt Furol at 122°F., sec		200–320								80–160	200–400		
Saybolt Furol at 140°F., sec			150–300	350–550		150–250	300–500	500–800				275–400	700–1400
Saybolt Furol at 180°F., sec									170–280				
Total distillate to 374°F., % by volume										5+			
Total distillate to 437°F., % by volume		2−	2−	2−	10−	2−	2−	1−	1−	12+	10+	3+	0.5+
Total distillate to 600°F., % by volume		15−	10−	8−	25+	10–20	8–20	16−	14−	25+	20+	14+	7+
Total distillate to 680°F., % by volume	50−	25−	20−	18−	50−	27−	25−	25−	20−	40−	35−	30−	25−
Float of residue, 122°F., sec	50−	25+	25+	25+									
Penetration of residue, 100 g., 5 sec., 77°F					70–300	100–300	100–300	100–300	100–300	60–120	60–120	60–120	60–120
Ductility of residue, 77°F., cm					60+	60+	60+	60+	60+	60+	60+	60+	60+
Solubility of residue in CS_2, %	99.0+	99.0+	99.0+	99.0+	99.5+	99.5+	99.5+	99.5+	99.5+	99.5+	99.5+	99.5+	99.5+
Recommended uses:													
Dust layer	X												
Primer					X								
Surface treatments										X			
Mixed-in-place construction:													
Aggregate, densely graded[2]		X	X[6]			X							
Aggregate, open graded[3]											X	X[6]	
Coarse aggregate[5]												X	
Plant-mixed construction:													
Aggregate, densely graded[2]			X	X			X						
Aggregate, open graded[3]											X	X	
Aggregate, open graded[4]								X	X				
Coarse aggregate[5]												X	X

[1] PAULS, J. T., "A General Outline of the Construction of Low Cost Bituminous Roads," *Roads and Streets*, Vol. 77, No. 4, p. 133, April, 1934.
[2] Maximum size not over 1 in. Fairly uniform grading from coarse to fine with appreciable percentage passing No. 200 sieve.
[3] Maximum size not over 1½ in. Fairly uniform grading from coarse to fine with little or no material passing No. 200 sieve.
[4] Maximum size not over 1 in. Fairly uniform grading from coarse to fine with little or no material passing No. 200 sieve.
[5] Maximum size not over 1½ in. Little or no material passing ¾-in. screen.
[6] Under favorable conditions only.

5. *Softening Point:* Ring-and-ball Method, A.S.T.M., D36-26, 1936 B.S., II, p. 1098; Cube-in-water Method, A.S.T.M. D61-24, 1936 B.S., II, p. 1103.—The fact that there are two softening-point tests is explained by the fact that the specific gravity of asphalts is so near unity, or even slightly less than unity for some materials, that it is necessary to weight the sample with a small ball in order to secure positive results. If the sample were not weighted, it might float instead of sinking. Tars have a sufficiently high specific gravity to give a positive test without a weight, and therefore the Cube-in-water Method is satisfactory. The test shows the temperature at which the material becomes sufficiently plastic to flow and is useful in determining the suitability of materials to be used for fillers. It is closely related to the Consistency Test but not a substitute for it, as most of the materials for which this test is used are too hard for the penetration test. In any case the penetration test cannot be employed for tars because of the free carbon content, which interferes with the test.

6. *Ductility,* A.S.T.M., D113-35, 1936 B.S., II, p. 1058.—The ductility test aids in the identification of some types of asphalt (the test is applied only to asphalts) and also aids in forming a judgment as to the quality of a new material. Blown oils and non-asphaltic residues have low ductility, usually less than 10 cm., and brittle materials often have a ductility in excess of 100 cm. The ductility of acceptable asphalt cements usually lies between 25 and 85 cm., although there are some exceptions.

7. *Solubility in Carbon Disulphide,* A.S.T.M., D4-27, 1936 B.S., II, p. 1041.—The percentage of bitumen, which is the constituent of the material useful as a binder, is determined by this method. The results are useful in two ways: (1) as an aid in identifying products made from tars or natural asphalts, each variety of which exhibits quite distinct characteristics in this respect; (2) as a means of calculating the quantity of asphalt cement to use in a mixer batch to secure the predetermined percentage of bitumen. If the mixture is to contain 7 per cent bitumen by weight, and the asphalt cement is 92 per cent bitumen, then the percentage of asphalt cement must be $7/0.92 = 7.61$. When this test is made on tars it also aids in identifying the type, as the gas-house tars run higher in free carbon than the coke-oven tars. Water-gas tars contain little free carbon.

8. *Bitumen Soluble in Carbon Tetrachloride,* A.S.T.M., D165-27, 1936, B.S., II, p. 1044.—Note that this test is for the percentage of *bitumen* soluble, not of the *asphalt cement* soluble. The bitumen in asphalt is almost equally soluble in carbon disulphide and carbon tetrachloride; and if this test shows more than about 2 per cent of the bitumen to be insoluble in CCl_4, there is something to be explained. It may indicate overheating in the fluxing or the addition of some unsuitable flux or residual asphalt. The portion of the bitumen *insoluble* in CCl_4 is called *carbenes.*

9. *Bitumen Insoluble in Standard Naphtha.*—This test is performed in accordance with the method adopted for determining the solubility in CCl_4, except that the solvent is paraffin naphtha of 88°Bé. gravity. The *insoluble* portion of the bitumen is called *asphaltenes.* The test is used primarily as an identification test, as some of the asphalts have very characteristic asphaltene percentages (note the analyses on page 336). The test is not widely used.

10. *Bitumen in Paving Mixtures.*—Although not so designated, A.S.T.M., D147-27, 1936 B.S., II, p. 1143, may be used for this determination. This test is routine in the inspection and control of asphalt plant operation and has for its purpose the determination of compliance with the specifications as to the bitumen content of the mixtures produced at the plant. The test is also useful in a variety of problems of proportioning bituminous mixtures and in studying the composition of natural rock asphalts.

11. *Flash and Fire Points*, A.S.T.M., D92-33, 1936 B.S., II, p. 892; or A.S.T.M., D93-36, 1936 B.S., II, p. 897; or A.S.T.M., D56-36, 1936 B.S., II, p. 907.—The particular method to employ depends upon the nature of the material to be tested, especially the temperature at which flash occurs, as is made clear in the several specifications listed. The test is employed to determine the temperature to which it is safe to heat the material in the mixing plant without danger of its catching fire. The temperature at which flash occurs is also an indication of that at which the product has been prepared, except in the case of cut-backs. This test is routine in the laboratory control of plant operation.

12. *Specific Gravity*, A.S.T.M., D71-27, 1936 B.S., II, p. 1008; and A.S.T.M., D70-27, 1936 B.S., II, p. 1110.—The test for specific gravity is largely employed as an aid in the identification of bituminous materials. When the specific gravity is considered in comparison with other properties of a material, it may afford exact evidence of the source of the materials The specific gravity must also be employed in all control computation. involving transposing from weight to volume or the converse.

13. *Residue of Specified Penetration*, A.S.T.M., D243-36, 1936 B.S., II, p. 1083.—Road oils are sometimes purchased on the basis of a specification as to the percentage of the oil that shall consist of asphalt of 100 penetration (sometimes a different penetration is specified, but 100 is standard). This test is routine in determining compliance with that specification. Purchasers sometimes specify that the residue of this or any other designated penetration shall comply with certain other requirements such as solubility in CCl_4, ductility, or specific gravity, and the residue obtained in the determination of percentage of a given penetration is available for other tests.

14. *Fixed Carbon.*—Fixed carbon is usually determined by the method prescribed for coal, A.S.T.M., D271-33, 1936 B.S., II, pp. 398–401.—The fixed carbon test is principally employed as an aid in the identification of residual asphalts and road oils. The asphalt base residues have higher percentages of fixed carbon than the paraffin base. The test is not sufficiently accurate to permit drawing conclusions from the results of the test unless supported by the results of other tests. For example, low specific gravity, low fixed carbon, and low ductility would be good evidence that the sample was from a mixed-base petroleum of doubtful stability as a road binder.

15. *Distillation of Road Tars*, A.S.T.M., D20-30, 1936 B.S., II, p. 1053.—The distillation test is employed for controlling the quality of tars used as a road material, and the test is routine in determining compliance with the specifications.

16. *Distillation of Creosote*, A.S.T.M., D246-33, 1936 B.S., p. 542.—This test is employed for controlling the quality of the creosote oils used for wood

preservation and the test is part of the routine of checking compliance with the specifications under which the material is purchased.

17. *Loss on Heating,* A.S.T.M., D6-33, 1936 B.S., II, p. 1071.—This is a test designed to show the rapidity with which asphaltic materials may be expected to harden when placed in service. It is an accelerated test that is very illuminating to an operator who can interpret the test not only in the light of the numerical results but also on the basis of the appearance and feel of the sample after the test has been completed.

18. *Stability of Asphalt-sand Mixtures.*—No national standard test has been devised for measuring the stability of asphalt pavement mixtures, but the Hubbard-Field[1] method is widely used and is admirably adapted for studying the effects on mechanical stability of variations in the grading of the mineral aggregates and in the proportions of bitumen in paving mixtures.

Estimating Quantity of Asphalt Cement Required.—The laboratory is frequently called upon to make preliminary estimates of the quantity of bitumen required for mixtures of mineral aggregates of known gradation. The following are a few of the more widely known of the empirical formulas employed in estimating the bitumen requirements from the sieve analysis data:

C. L. McKesson and W. N. Frickstad, California:

$$P = 0.015a + 0.03b + 0.17c,$$

where
- P = percentage of oil by weight.
- a = percentage of material retained on 10-mesh.
- b = percentage of material passing 10 and retained on 200-mesh.
- c = percentage of material passing 200-mesh.

New Mexico:

$$P = 0.02(a) + 0.07(b) + 0.15(c) + 0.20(d),$$

where
- P = percentage of liquid asphaltic material required.
- a = percentage of aggregate between 50-mesh sieve and maximum size.
- b = percentage of aggregate between 50- and 100-mesh sieve.
- c = percentage of aggregate between 100- and 200-mesh sieve.
- d = percentage of aggregate passing 200-mesh sieve.

Nebraska:

$$P = AG(0.02a) + 0.06b + 0.1c + Sd,$$

[1] This test is described in "The Rational Design of Asphalt Paving Mixtures," *Research Ser. Pamphlet* 1, The Asphalt Institute, New York, N.Y., p. 6, Nov. 1, 1934.

where

P = percentage bitumen by weight at the time the bituminous mixture is laid.

A = absorption factor for aggregate (for hard gravel, granite, or other similar materials this factor is equal to unity).

G = Gravity correction factor (computed by dividing the bulk specific gravity of the aggregate to be used into 2.62 which is the bulk specific gravity of Platte River gravel).

a = percentage of the aggregate retained on the No. 50 sieve.

b = percentage of the aggregate passing the No. 50 sieve and retained on the No. 100 sieve.

c = percentage passing the No. 100 sieve and retained on the No. 200 sieve.

d = percentage of aggregate passing the No. 200 sieve.

S = a factor the value of which depends upon the character and absorptiveness of the portion of the aggregate passing the No. 200 sieve.

Michigan:

$$r = k + 0.041y$$

where

r = pounds of oil per 100 lb. of aggregate with specific gravity of 2.65.

k = constant depending upon shape and absorption of aggregate.

y = surface-area variable depending on grading of aggregate.

The constant (k) is determined for various aggregates by laboratory tests ($k = 2.71$ for average Michigan aggregates). For average Michigan aggregate:

$$r = 2.71 + 0.041y.$$

The surface area of any grading is determined as follows:

Percentage retained on No. 10 × 4.	=
Percentage passing No. 10 retained on No. 40 × 18	=
Percentage passing No. 40 retained on No. 200 × 80	=
Percentage passing No. 200 × 250[1].	=
Surface area (y).	total.

The oil ratio (r) is corrected for specific gravity of aggregate as follows:

$$R = \frac{2.65r}{\text{Specific gravity of aggregate}}$$

[1] The surface area factor for material passing the 200-mesh sieve is somewhat of a variable depending upon absorption; however, for average silts and commercial fillers a factor of 250 is satisfactory while other fillers such as marls, some silts, and sugar beet lime may require a factor as high as 1,000. Each filler is tested and proper factor selected to correct for absorption characteristics.

where

R = pounds of oil per 100 lbs. of aggregates used

The percentage of oil is determined from formula.

$$\text{Percentage of bitumen} = \frac{100R}{100 + R}.$$

Wisconsin:

I—*Formula for Mixes Containing a Predominance of Material Retained on #4 Sieve:*

$$\text{Per cent bitumen by weight} = A \times S \times F(0.05R_{3/4} + 0.12R_4 + 0.04P_4).$$

II—*Formula for Mixes Containing a Predominance of Material Passing #4 Sieve:*

$$\text{Per cent bitumen by weight} = A \times S \times F(0.07R_{30} + 0.1R_{100} + 0.45P_{100})$$

A = absorption factor.
S = specific gravity factor.
F = film coefficient factor.
$R_{3/4}$ = percentage of aggregate passing 2 or 1½-in. sieves and retained on the ¾-in. sieve.
R_4 = percentage of aggregate passing the ¾-in. sieve and retained on the No. 4 sieve.
P_4 = percentage of aggregate passing the No. 4 sieve.
R_{30} = percentage of aggregate passing 1½-in. sieve and retained on the No. 30 sieve.
R_{100} = percentage of aggregate passing No. 30 sieve and retained on the No. 100 sieve.
P_{100} = percentage of aggregate passing No. 100 sieve.

Analyses of Commercial Bituminous Materials.—The following analyses will indicate the character of some of the bituminous materials now in use and will show how the various tests described on page 358 are employed to secure a record of the nature of the materials that are utilized for specific construction projects.

The following analyses indicate the general character of some of the commercial bituminous materials now being used:

DUST LAYERS FOR EARTH ROADS

	No. 1	No. 2
Specific gravity at 25°C.	0.94	0.93
Specific viscosity (Engler) 50 cc. at 50°C.	12	14
Total bitumen	99.86 per cent	99.99 per cent
Fixed carbon	2.16 per cent	6.8 per cent
Volatilization loss on 50 g., 5 hr., 163°C.	0.92 per cent	11 per cent

DUST LAYERS FOR GRAVEL AND MACADAM ROADS

	No. 1	No. 2
A. Tar:		
Specific gravity at 25°C.	1.13	
Specific viscosity on 100 cc. at 40°C.	24.6	
Free carbon (insoluble in benzol)	5.2 per cent	
Distillation fractions by weight:		
Total distillate up to 170°C.	1.4 per cent	
Total distillate up to 270°C.	28.0 per cent	
Total distillate up to 300°C.	31.0 per cent	
B. Petroleum Oil:		
Specific gravity at 25°C.	1.01	1.04
Consistency by float test at 50°C.	88 sec.	18 sec.
Total bitumen	99.7 per cent	99.9 per cent
Asphaltenes	19 per cent	
Fixed carbon	11.2 per cent	8.58 per cent
Volatilization loss 50 g., 5 hr., 163°C.	2.01 per cent	7.4 per cent
C. Tar for Gravel-road Maintenance:		
Specific viscosity, 50 cc. at 40°C.	....	9.9
Free carbon	....	4.04 per cent
Distillation fractions by weight:		
0 to 170°C., 1.0 per cent		
0 to 270°C., 23.3 per cent		
0 to 300°C., 31.8 per cent		
Softening point of residue for distillation, 49.6°C.		

Carpeting Mediums and Binders for Mastic

A. Tar:	
Specific gravity at 25°C	1.22
Specific viscosity (Engler) 100 cc. at 100°C.	14
Free carbon (insoluble in benzole)	13.6 per cent
Distillation fractions by weight:	
Total distillate up to 170°C	0.0 per cent
Total distillate up to 270°C	15.0 per cent
Total distillate up to 300°C	20.6 per cent
B. Asphalt:	
Specific gravity at 25°C	0.985
Float test at 50°C	120 sec.
Penetration at 25°C., 50 g., 5 sec. at least	350
Loss at 163°C., 5 hr	7 per cent
Total bitumen	99.5 per cent
Fixed carbon	6 per cent
Percentage bitumen insoluble in 86° naphtha	15 per cent

Binders for Penetration and Mixed Macadam

A. Tar:	
Specific gravity at 25°C	1.24
Melting point, cube-in-water method	50°C.
Free carbon (insoluble in benzole)	17 per cent
Distillation fractions by weight:	
Total distillate up to 170°C	0.0 per cent
Total distillate up to 270°C	8.0 per cent
Total distillate up to 300°C	12.0 per cent
B. Fluxed Native Bitumen:	
Specific gravity at 25°C	1.054
Consistency by penetration method at 25°C., 100 g., 5 sec	95.6 per cent
Asphaltenes	26.6 per cent
Fixed carbon	11.1 per cent
Volatilization loss, 50 g. at 163°C	4.26 per cent
Ductility	48 cm.
C. Petroleum Product:	
Specific gravity at 25°C	1.03
Consistency—penetration at 25°C., 100 g., 5 sec	100
Total bitumen	99.9 per cent
Asphaltenes	21 per cent
Fixed carbon	16 per cent
Volatilization loss 50 g., 5 hr., 163°C	0.12 per cent

Analyses of Asphalt Cements

	"Texaco"	"Stanicolo"
A. Petroleum Products:		
Specific gravity at 25°C.	1.03	1.042
Consistency by penetration at 25°C., 100 g., 5 sec	54	56
Total bitumen	99.9 per cent	99.9 per cent
Asphaltenes, insoluble in 86°B., naphtha	25 per cent	23 per cent
Fixed carbon	11.6 per cent	16.7 per cent
Melting point, Ring and Ball	140°C.	132°C.
Ductility	75 cm.	100 cm.
Flash point	250°C.	270°C.
Volatilization loss on 50 g., 5 hr., 163°C	0.06	0.055 per cent
	"Bermudez"	"Trinidad"
B. Fluxed Native Bitumens:		
Specific gravity at 25°C.	1.045	1.24
Consistency by penetration at 25°C.	55	58
Total bitumen	96.2 per cent	74 per cent
Asphaltenes, insoluble in 86°B., naphtha:		
Ductility	32 cm.	28 cm.
Fixed carbon	12.6 per cent	16.9 per cent
Volatilization loss 50 g. at 163°C	0.5 per cent	0.7 per cent

Petroleum Flux

Specific gravity at 25°C.	1.008
Consistency by float test at 50°C.	185 sec.
Total bitumen	99.8 per cent
Volatilization loss, 50 g. at 163°C	0.4 per cent
Flash point (open cup)	190°C.

FILLERS FOR BLOCK PAVEMENTS

	No. 1	No. 2
A. Petroleum Asphalt:		
Specific gravity	1.015	1.044
Melting point, Ring and Ball	69°C.	57°C.
Penetration at 25°C.	32	41
Volatilization loss 50 g., 163°C.	0.7 per cent	0.02 per cent
B. Tar:		
Specific gravity at 25°C.	1.22	
Free carbon	24 per cent	
Melting point, cube-in-water method	135°F.	
Distillation fractions by weight:		
Total distillate up to 170°C.	0	
Total distillate up to 270°C.	2 per cent	
Total distillate up to 300°C.	5 per cent	

CHAPTER XV

DUST-LAYING TREATMENTS AND BITUMINOUS CARPETS

Bituminous treatments intended merely to suppress dust are not infrequently employed, and successive applications for that purpose may eventually build up a thin surface layer which is called a "bituminous mat" or a "bituminous carpet." Sometimes the construction is undertaken with the intention of securing a mat at once because of its superior value as a preventative of dust and also because it affords a stable, waterproof surface.

DUST SUPPRESSION ON EARTH ROADS AND STREETS

Earth roads (natural soil surfaces in general) are sometimes oiled to suppress the dust and, incidentally, to waterproof them. This treatment is recognized as no more than a palliative but, in some locations, worth the cost.

Road Oils.—It has been noted (page 83) that when a soil contains the most favorable (optimum) quantity of moisture, the particles are bound together into a fairly stable mass by the surface tension of a film of water. An earth-road surface loses moisture rapidly in dry weather and if the soil is of the groupings *A*-1, *A*-2, or *A*-3 (page 102) will be very dusty in continued dry weather. The *A*-4 and *A*-5 types do not become disagreeably dusty except under very heavy traffic or long-continued dry weather. The function of a road oil is to supply a liquid of low volatility the inherent surface tension of which is great enough to bind the particles of soil together. The characteristics most essential to a good road oil are low volatility and high surface tension, and these militate against the penetration of the oil into the road surface. The road oil must be of such a composition that it will soak into the soil; therefore it must be of high viscosity (very liquid) and cannot carry more than 35 to 50 per cent of asphalt. The oils used for dust suppression on earth roads are called "road oils," "fuel oils," "liquid asphalt," and the like, but

their essential characteristics are those of S.C.1, Table XXVI, except that the asphalt content is lower, and the oil has a higher viscosity. No general specifications for these road oils are available.

Preparation for Oiling.—If a road or street is to be oiled for the first time, preparations should be started some weeks before the oil is actually applied.

The effect of surface oiling is to render the earth partially impervious to moisture; and if the road is uneven when oiled or becomes uneven afterward, the depressions will serve as waterproof basins for holding water. Traffic will gradually work the soil and the water into mud, deepening the depressions and hastening the deterioration of the surface. If the surface is smooth and well-crowned, the water will run to the gutters so quickly that only in long-continued wet weather will the surface be softened sufficiently to get muddy.

The principal object of oiling is to prevent dust; therefore, there should be no dust on the street when the oil is applied. What is more important, oil does not readily penetrate a layer of dust. If dust has formed, it should be removed before oiling begins, which costs something. It is therefore cheaper to treat the surface before the dust has formed than to remove the dust as a preliminary to oiling.

For the best results in humid regions, oiling should be planned ahead, and the preparation be carried out in the early summer so that the oiling can be done before a layer of dust forms on the street with the arrival of dry weather. In dry regions the oiling may be undertaken at any time of the year if the temperature is high enough to keep the oil fluid until it is absorbed into the soil.

The decision to oil a road surface is sometimes reached after the roadway surface has become hard and dry. In that event, it is not advisable to do any extensive earthwork, because in continued dry weather newly placed earth will not compact; and if oiled before being well packed, the results are unsatisfactory.

Applying Oil.—The oil is applied at a rate of 1/3 to 1/2 gal. per square yard of surface. If the surface has never been oiled before, or if more than a season has elapsed since a previous oiling, it will usually be necessary to use about 1/2 gal. per square yard; but if the surface has been oiled regularly each season, about 1/3 gal. per square yard is generally sufficient after the first

year. For continued dust suppression it is necessary to oil every year, and towns having unpaved business streets must expect to oil them twice a year. In business areas the oiling must be done by sections to permit traffic to have access to the area.

After the oil has been spread, it is allowed to stand for a day or so to permit it to soak into the surface. It is then covered with just enough sand to keep the oily earth from adhering to the wheels of vehicles. Emphasis is placed on the importance of employing sand for this purpose rather than dust from the road surface. The amount of sand needed is small, and at any reasonable price the benefits derived justify its use when it can be secured.

After the road has been put into service it may be apparent that more sand is needed in spots to prevent vehicles from picking up the oil, and such places should be covered lightly, the operation being repeated two or three times if necessary.

When a roadway is oiled the second time, the method to be followed is exactly the same as is followed the first time, except that the quantity of oil may be reduced to about $\frac{1}{3}$ gal. per square yard of surface. It is advisable to repeat the oiling the second year in any case, but it may be omitted the third year and resumed the fourth year. Better results would be obtained if the treatment were repeated every year.

Results to Be Expected.—Surface oiling forms a layer of granular or finely powdered soil which is oil-saturated and consequently does not blow about readily. The suppression of dust is the principal benefit to be expected. Beneath the thin layer of oil-soaked surface the soil is partially saturated with oil to a depth that varies from 1 to perhaps 6 in.

Water penetrates an oiled roadway rather slowly. If the surface has enough cross-slope to free itself of surface water, only a small amount of mud will form under light or moderate traffic. A roadway that is oiled systematically for a series of years gradually acquires an oil-soaked crust which becomes more and more impervious as the oiling is repeated but never reaches a degree of saturation that will prevent it from becoming muddy in seasons of heavy rainfall, nor will it be stable in ordinary wet weather if the road carries heavy traffic.

Handling Road Oils.—Road oils are delivered to construction projects in tank cars which are equipped with steam coils for heating the oil when that is necessary. Contractors and main-

tenance departments have portable pumping plants which can be used for pumping the oil from the tank cars to the distributors used for spreading the oil on the roads. These plants have sufficient capacity to load the distributor tanks quite rapidly and also furnish steam to heat the oil in the cars to facilitate handling.

The oils are distributed from tank trucks which are eauipped with pumps that take the oil from the truck tank and force it through a distributor pipe to nozzles which spray the oil on the road surface. Reasonably accurate adjustments of the quantities distributed are possible with these mechanisms, and the truck tanks can cover an enormous yardage daily.

Cost of Surface Oiling.—The total cost of oiling earth roads or streets will vary between the following limits, assuming a treatment of ½ gal. per square yard of surface, and the cost of oil to be 4 cts. per gallon.

TABLE XXVII.—COST PER SQUARE YARD FOR OILING

Item	Minimum	Maximum	Average
Cleaning surface	0.0025	0.0050	0.00375
Hauling and distributing oil and sanding	0.0075	0.0125	0.0100
Oil	0.0300	0.0600	0.0450
Sand	0.0025	0.0075	0.0050
	0.0425	0.0850	0.06375

DUST SUPPRESSION ON GRAVEL AND MACADAM

The use of bituminous materials for dust suppression on gravel and macadam road surfaces is limited to those cases where these types of surfaces are capable of sustaining the loads imposed and carrying the volume of traffic without undue wear but become disagreeably dusty in periods of prolonged dry weather. The object of the bituminous treatment is to mitigate the disagreeable dust of the ordinary gravel or macadam road without incurring the expense that would be required to apply a long-wearing type of bituminous surface. Since dust is always an accompaniment of wear, any treatment that will suppress the dust must of necessity reduce materially the rate of wear of the surface.

Origin of Dust.—The dust from the ordinary gravel road results primarily from the loosening of the soil mortar that constitutes

the filler between the pebbles of the surface. When this fine material is being loosened and whipped off the surface in the form of dust, an objectionable amount of surface deterioration is taking place at the same time. Gravel roads that are subjected to moderate traffic may be disagreeably dusty, without the loss of binder in the form of dust being particularly significant from the standpoint of maintenance of the road. The dust from the untreated gravels and macadams may constitute a serious nuisance to the residents along the road as well as to the traffic. This is particularly true of gravel roads in the outskirts of municipalities or in summer resort districts; of gravel streets in the smaller towns, especially residential streets; and occasionally on sections of the main county roads.

The dust from the macadam types of surfaces consists of fine particles of rock ground off the metal of the road and of the screenings dislodged by vehicle tires from the void space between the larger stones. This fine material constitutes the binding element in the surface, and its loss in any considerable quantity presages serious deterioration of the road surface.

Bituminous Materials for Dust Suppression.—It was pointed out that the type of dust layer to employ on an earth road was an oil that would have a very fluid consistency and could be expected to soak into the road surface. Bituminous materials that would have the requisite stability cannot penetrate the gravel or macadam surface to the extent that is possible with the road oils used in treating the earth road. A gravel road consists of pieces of rock, which will not absorb oil, bound together with soil mortar which is similar in character to the soil that constitutes an earth road. Therefore, on a graveled road materials should be employed for dust-laying purposes that will have about the same fluidity as those employed for dust laying on earth roads. These bituminous oils should, however, be more viscous after they have cured than the road oils, and that means that they should carry a somewhat heavier asphalt cement than the road oils, cut back, however, until it is sufficiently fluid at air temperature at the time of application to penetrate the surface being treated.

The binding material between the individual stones in a macadam road is stone dust, and the void space in such a surface is entirely different from that of a gravel road. The voids are larger than those found in the soil-mortar binder of the gravel road and are more irregularly distributed over the surface. A

considerable percentage of the surface area consists of the faces of the individual pieces of rock that constitute the road metal. The rock itself is not susceptible to penetration by any type of binder any more than the pebbles in gravel. Consequently the dust layer for the macadam road should be a material that will adhere to road metal and at the same time be cut until it has sufficient fluidity to penetrate the relatively coarse voids in the filler between the pieces of road metal. The material must harden promptly after it is placed on the road, or a great deal of it will flow to the slight depressions in the road surface and fail to cover adequately the pieces of rock.

For the binder for dust-laying purposes on graveled roads the types of material most widely used are those of S.C.1 or M.C.1 in Table XXVI; and for dust-laying purposes on macadam materials, such as R.C.1 or M.C.2 in Table XXVI, would probably be used.

Preparation of Surface to Be Oiled.—The preparation for a dust-laying application to a graveled road begins some weeks prior to the actual application of the bituminous material and consists in carefully shaping the surface with the ordinary maintenance equipment to bring it to an even contour with very little cross-slope. In humid areas this work begins during the season of the year when there is considerable rainfall and the road surface is accordingly wet, or at least moist, a good deal of the time. Just prior to applying the dust layer the excess loose material is scraped and swept off the road surface. However, if the preparation of the surface has been carried out intelligently, and the oil is applied at the most timely period in the year, it will be found that traffic has put the road surface into an ideal condition to receive the oil. In preparing for this kind of work it is recognized that any pockets of fine dust on the surface will absorb the bituminous material very slowly, if at all; and men are sent along with brush brooms to remove any little patches of dust that have accumulated in small depressions in the road surface and have escaped removal by the ordinary blade machine.

The preparation of the macadam road is somewhat similar to that described for gravel except that macadam roads are not maintained with a blade grader but, on the contrary, are maintained by patrolmen who patch the breaks in the road surface with hand tools. Just before the oiling is to be undertaken the

excess fine material is removed from the macadam surface by sweeping with a power broom, followed by hand sweeping to remove the patches of dust that escape the power broom. The object is to expose as much as possible of the road metal. Some types of macadam are constructed with stone that is so soft that traffic pounds the surface stone into a powdery form, and it would be impossible to remove all the fine material from such a surface by any practicable method of cleaning; in fact it would largely destroy the texture of the road surface. In the case of these very dusty types of macadam roads the preparation for the oiling is about the same as the preparation of a gravel road, and the type of oil must be about the same.

Gravel roads consisting of a large proportion of coarse stone and a minimum of binder, a type that is found extensively in some localities, have a surface that is very much like the macadam, and the maintenance and preparation for oiling are identical with those described for the broken-stone macadam.

Application of Oil.—The oil is applied to the surface by means of the pressure distributors that are commonly used in all types of bituminous application, and for the graveled road the amount of oil applied ranges from $\frac{1}{3}$ to $\frac{1}{2}$ gal. per square yard. It will require several days for the oil to penetrate the road surface, during which time traffic cannot be permitted without seriously interfering with the effectiveness of the treatment. The oil should be of such a type that at the end of a reasonable curing period, traffic can be allowed without danger of oily gravel sticking to the wheels. If the surface proves to be somewhat sticky after the curing period, it must be lightly dressed with sand and dust from the shoulder alongside the road.

The bituminous material is applied to a macadam road in about the same manner as described for the gravel road, except that the quantity is somewhat less because the macadam surface absorbs less oil than does the gravel surface. Usually the application to macadam will range between $\frac{1}{4}$ and $\frac{1}{3}$ gal. per square yard. For these surfaces it is desirable to have a type of oil that hardens rather rapidly so that it will become stable before it has had a chance to run off the unabsorbent part of the road. It will be found with bituminous materials of the type used for macadam and coarse gravel surfaces that a light dressing of coarse sand or fine stone chips will be advantageous following the application of the oil.

Subsequent surface treatment should be made to these surfaces after an interval of 6 weeks or 2 months during the first season that the dust-laying process has been employed. After that, treatments need not be made oftener than once in 2 years unless the traffic is very heavy, in which case annual oiling will be necessary. It is more likely, however, that if the traffic is heavy, a bituminous mat will be employed instead of the conventional dust-laying treatment.

General Results.—The effect of bituminous oils on graveled surfaces is to saturate the fine material that constitutes the binder with a material containing some very soft asphalt. Upon evaporation of the volatile constituents of the road oil a soft asphalt remains which serves to bind the dust particles together and thus prevent their being blown about by the action of traffic. As a matter of experience the treatment is seldom wholly effective, and some fine brown dust will be found on the road of this type after a certain interval following oiling; there is no way to prevent this except by applying a bituminous carpet to the surface. However, oiling is resorted to only on those roads of moderate traffic where it seems desirable to go to the expense of a treatment to suppress the dust but where construction of the more durable carpets or mats would not be justifiable.

The dust-suppression treatment on macadam roads is somewhat more satisfactory than on the gravel, as a rule.

The cost of these dust-suppression treatments on gravel and macadam ranges from $500 to $1,000 per mile, depending upon the length of haul for materials and the amount of special work in connection with the preparation of the road surface. After the first year's oiling, subsequent applications can be made for a somewhat smaller sum because of the lessened cost of preparing the road surface for the oil. In any case, however, the application of bituminous oils to these surfaces as a dust layer must be looked upon as an expedient that should be undertaken only when there are insufficient funds to permit the construction of some other type of thin bituminous surface that will be more durable, or where the cost of these more durable surfaces is greater than is warranted in view of the benefits. There are many cases, however, in which the dust-laying treatment is entirely justifiable.

Repeated applications of road oils of good quality as dust palliatives may result in the gradual accumulation of a thin

layer of asphalt and sand or stone chips which has all the characteristics of a bituminous mat such as is described in a later section. That is, a mat has been constructed by stages over a period of several years.

Other Dust Preventives.—Several proprietary compounds not of a bituminous character have been used from time to time as dust preventives. The most widely used have been the hygroscopic salts and certain semisticky by-products of industrial processes which have been applied in water solutions. None of these palliatives is of permanent value; but calcium chloride, for example, is effective for a considerable period when applied to gravel roads.

Water sprinkling is the conventional method of dust suppression in the smaller towns and needs no discussion. It is effective but messy and expensive.

BITUMINOUS MATS ON GRAVEL AND MACADAM

Bituminous mats are wearing surfaces consisting of bituminous cement, mineral aggregate of a size that will pass a ½-in. screen, often called "chips," bonded into a stable mat by means of bituminous cement and held to the surface of the underlying gravel or macadam by adhesion and the interlocking of the angular fragments of rock in the surface and base. The thickness of these mats is rarely more than ½ in. and frequently is less.

Binder for the Bituminous Mat.—The bituminous mat as generally constructed requires two kinds of bituminous material, the one first applied being a priming coat, and the second being the cementing agent for the mineral aggregate.

Priming Coat.—The priming coat is intended to saturate the upper ½ to 1 in. of the old road surface to insure that the mat will adhere to the underlying material. The priming requires a penetrating material, and therefore oils of the type of S.C.1 or M.C.1 of Table XXVI will be used.

Binding Material.—The bituminous binder for the mat must be soft enough when applied to coat the mineral aggregate during the simple operation of spreading the mineral aggregate over the surface. It must contain an asphalt cement that will be adhesive and stable at the end of the necessary curing period. Materials like M.C.2 and R.C.2 or R.C.3 of Table XXVI will generally be used.

Mineral Aggregate for Bituminous Mats.—The mineral aggregate (chips) for bituminous mats is preferably crushed stone of a size that will pass a ½-in. screen from which most of the dust has been removed. Reasonable success has attended the use of gravel pebbles of the proper size, but in general this material is not so satisfactory as the chips from crushed stone. Mixtures of chips and coarse sand and even coarse sand alone have been employed with some success.

Construction of Bituminous Mats.—When a bituminous mat is to be constructed, the operation is identical with that of apply-

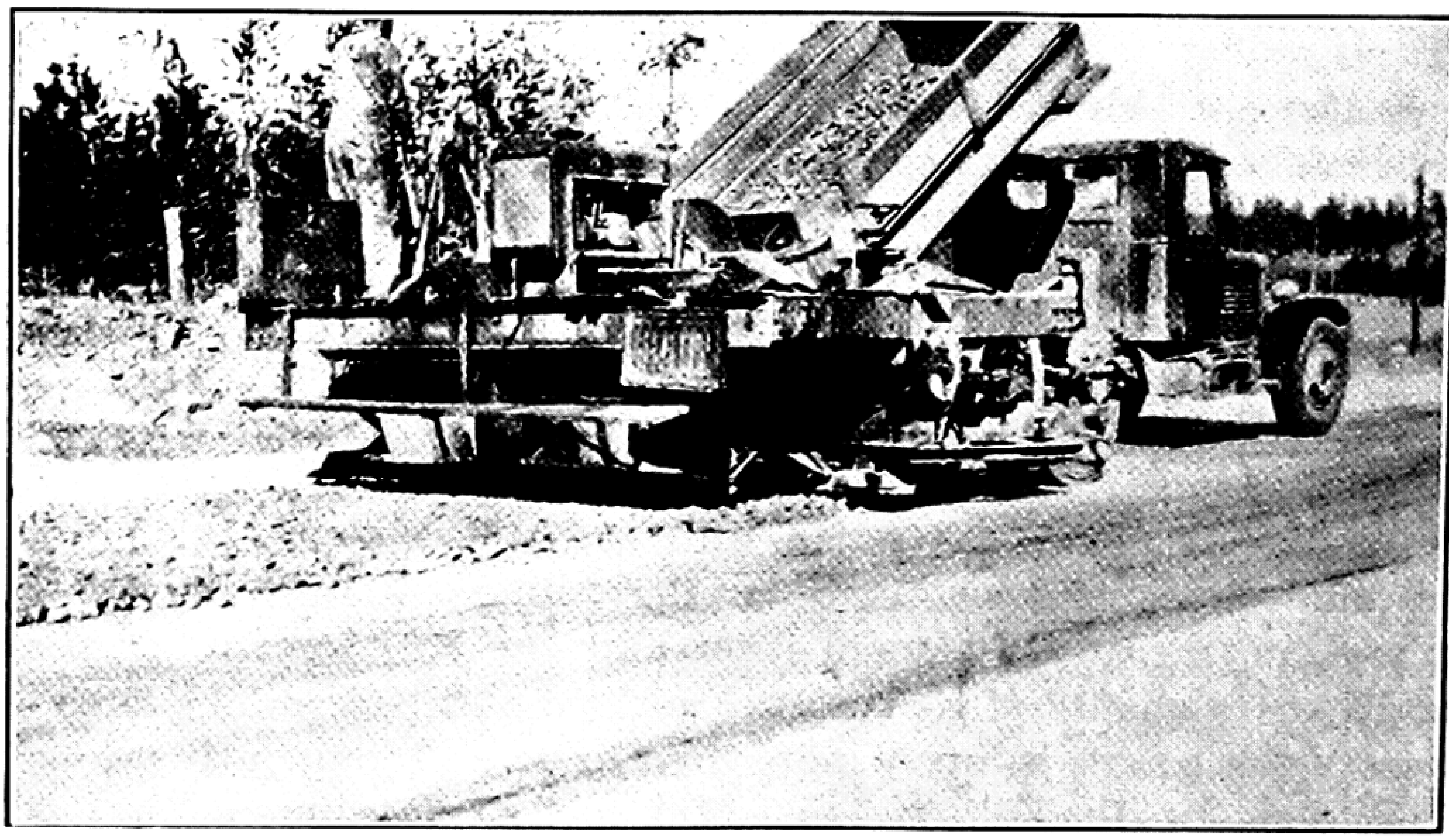


FIG. 94.—Spreading stone for a bituminous mat.

ing a dust palliative up to and including the application of the first coat of bituminous material. This first oiling serves as a priming coat; and when it has cured sufficiently to permit the distributor tanks to travel over the surface, the second coat of bituminous material is applied at the rate of about ⅓ gal. per square yard of surface. This coating is immediately covered with the mineral aggregate, generally by backing a truck along the surface so that it will travel over the portion that has already been dressed.

The mineral aggregate is carefully brushed over the surface by means of drags or maintenance machines to make sure that all parts are covered; and after the bituminous material has hardened sufficiently (usually within 48 hr.), traffic is allowed on the surface.

During the first days of travel occasional breaks will occur in this type of surface, and patches of the material will peel off because of poor bond to the priming coat; consequently, patrol maintenance is very important at this stage of the life of the surface. In a few weeks the mat will smooth out into a black sheet of granular texture which is quite durable when the bituminous material is fully cured. These mats often have a life of 2 or 3 years even on heavily traveled roads. They may be restored by the construction of another mat on top of the worn surface or by the addition of one of the types of mixed bituminous macadam surfaces.

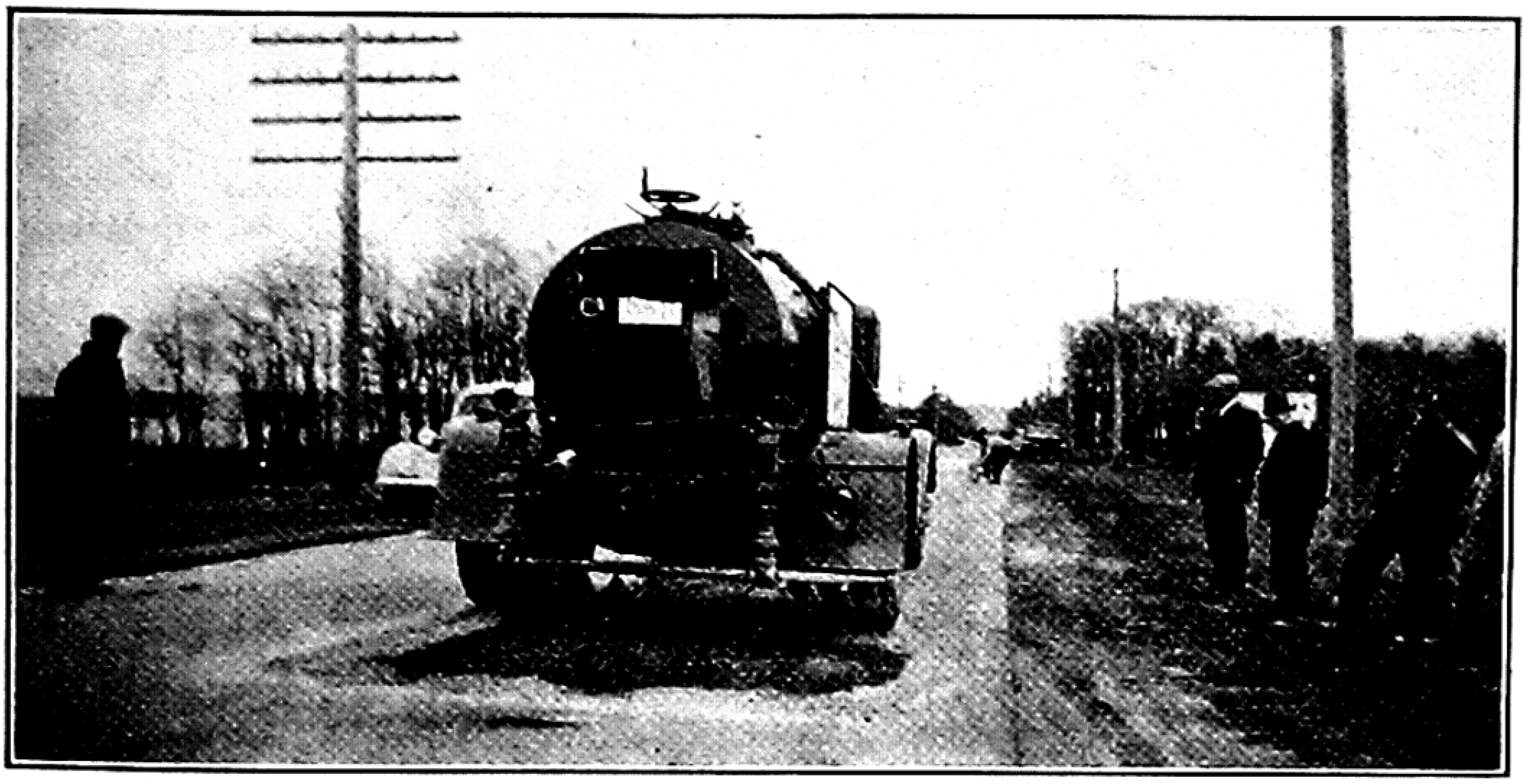


FIG. 95.—Spreading tar for a tack coat.

This type of surface is sometimes called "inverted penetration" construction, for reasons that will be apparent after a consideration of Chap. XVI.

BITUMINOUS MATS ON TREATED EARTH AND SAND-CLAY ROADS

In recent years extensive studies[1] have been made of the possibility of eliminating the loss of material from the surface of treated earth and sand-clay roads in dry weather, a loss that creates a serious dust nuisance and hastens the deterioration of the surface. On account of the fine-grained nature of the material composing the wearing surface of these roads, considerable difficulty has been experienced in securing a treatment that will actually stick to the surface. However, repeated trials have developed methods

[1] "Bituminous Surface Treatments of Sand-clay and Topsoil Roads," *Public Roads*, Vol. 10, No. 10, p. 207, January, 1930.

that have proved reasonably satisfactory on a considerable mileage of this type of construction. The problem here is not unlike that of suppressing the dust on earth roads.

Bituminous Materials for Surfacing Sand-clay Roads.—The problem involved in this particular type of construction is to find a bituminous material for a priming coat that will penetrate the sand-clay surface to a depth of 1 in. or more before hardening so much that penetration ceases. It is still further required, of course, that after the volatile part of the oil has evaporated, there remains sufficient bituminous cement of good binding properties actually to hold the road surface intact. Trials of various types

FIG. 96.—Surface dressing a bituminous mat.

of oil have uncovered certain materials that have proved satisfactory in a considerable mileage of experimental construction. On pages 356–367, paragraphs *D*, *E*, and *F*, are given specifications for materials that were used successfully for the priming coat and for the surface coat of roads of this type. Doubtless other materials of similar character will prove satisfactory.

Methods of Construction.—The first step in the construction is the application of the priming coat of bituminous material which, experience indicates, will require anywhere from a $\frac{1}{3}$ down to $\frac{1}{5}$ gal. per square yard of surface. The primer should be allowed to soak into the road surface for as long a period as may be required for all of it to be absorbed, so that it is possible to travel over the surface with the tank wagon in making the application of the heavier material for the wearing surface. The

second application of bituminous material will consist of $\frac{1}{3}$ to $\frac{1}{2}$ gal. per square yard of the bituminous binder selected, and the material should be of such a character that it can be applied at temperatures around 350°F.

As soon as the second application of bituminous material has been completed the surface of the road is covered with a layer of granular material, preferably chips, of a size that will pass a $\frac{3}{4}$- or 1-in. screen. The amount of this material required ranges from 100 to 150 lb. per gallon of bituminous material, or from 5 to $7\frac{1}{2}$ lb. of material per square yard of surface. This cover material should be angular granular material, that is, crushed rock or crushed gravel. Rounded pebbles are not a satisfactory cover for this type of road surface.

The bituminous material is applied to the road surface by means of the pressure distributor commonly used in bituminous construction. Stone chips are applied by broadcasting by hand from windrows alongside the treated surface. It is not easy to secure uniform distribution of the surface dressing in this way, and considerable brushing is required during the early weeks of the use of the road to secure uniform distribution of the loose material over the surface. Further experimenting with methods of construction for this type of surface will undoubtedly show that the mineral aggregate for the surface dressing can be applied to the surface by the mixed-in-place or the inverted penetration method which has been employed for graveled surfaces (page 379).

Mineral Aggregates for Surface Dressing.—The following specifications will indicate in general the character of the mineral aggregate that has been found satisfactory for the bituminous mats on the treated earth and sand-clay roads:

1. *Coarse Mineral Aggregate.*—The coarse mineral aggregate used for cover for the hot asphalt shall consist of approved slag or broken stone free from dust, thin or elongated pieces, dirt, or other objectionable matter occurring either free or as a coating on the aggregate. Stone from which it is produced shall have a percentage of wear of not more than 6 and a toughness of not less than 5. The coarse aggregate shall meet the following requirements:

	Per Cent
Retained on a $1\frac{1}{2}$-in. screen	0
Passing a $1\frac{1}{4}$-in. screen	90–100
Passing a $\frac{3}{4}$-in. screen	25– 75
Passing a $\frac{1}{4}$-in. screen	Not more than 10

2. *Fine Mineral Aggregate.*—The fine mineral aggregate used for cover for seal coat shall conform with either of the following sets of requirements:

a. Crushed Limestone: The crushed limestone shall have a percentage of wear of not more than 5 and a toughness of not less than 8. The material shall meet the following grading requirements:

	Per Cent
Passing ⅝-in. screen	100
Passing ¼-in. screen	Not more than 7

b. Crushed Granite: The crushed granite shall have a percentage of wear of not more than 5 and a toughness of not less than 8. It shall meet the following grading requirements:

	Per Cent
Passing ¾-in. screen	100
Passing ½-in. screen	25–75
Passing ¼-in. screen	Not more than 25
Passing 10-mesh sieve	Not more than 3

In some instances it has been found advantageous to make a third application of bituminous material to this type of road surface and cover it with fine mineral aggregate (as specified above), thus affording a seal coat of fairly dense construction. When such a seal coat is applied, the bituminous material is of the same character as that employed for the second application, the amount per square yard varying from ¼ to ⅓ gal. per square yard of surface. The amount of fine mineral aggregate required for cover material required is about the same as that which is employed for the covering of the first wearing course of bituminous material. Specifications for both types of mineral aggregate to use for cover are given above.

CHAPTER XVI

PENETRATION MACADAM AND BITUMINOUS CONCRETE

The types of surfaces known as bituminous macadams consist of those built by the penetration method and several varieties of bituminous concrete, characterized by the nature of the materials utilized and the methods of construction. They are treated together because of certain fundamental considerations that apply alike to all of them.

PENETRATION MACADAM

The name *penetration macadam* is given to a type of construction in which a bituminous material is sprayed on the surface of a layer of stone that has been prepared in such a manner that the bituminous material will penetrate 1 or 2 in. into the layer, thereby serving as a binder to hold the mass together. With a final dressing of chips, a thin mat is provided to cover the coarse aggregate of which the body of the road is composed, thus affording a satisfactory seal against the entrance of water into wearing surface.

Bituminous macadam may be constructed by the penetration method with any of the kinds of stone that are used for the ordinary macadam, although the methods of construction vary slightly with the character and size of the stone used.

Design of Cross-section.—Since the penetration macadam does not possess any considerable flexural strength, it supports traffic loads by distributing the load over the subgrade to such an extent that the bearing power of the soil is not exceeded (page 101). In this respect the penetration surface is analogous to the water-bound macadam. The design for any location is therefore based on engineering judgment and experience and on an appraisal of the load-carrying capacity of the subgrade upon which the wearing surface will be placed. The penetration surface is usually about 2½ in. thick. The lower, or base, courses vary in thickness and number according to the nature of the subgrade and the kind of traffic.

When used on city streets, penetration macadam is ordinarily constructed between combined concrete curb and gutter.

The cross-slope most frequently adopted is an average of ¼ in. per foot of width.

Typical cross-sections for penetration macadam are shown in Fig. 97.

Foundation Course.—The foundation course may consist of Telford with a layer of water-bound macadam over it, or it may consist of a layer of water-bound macadam placed directly on the subgrade. In either case the macadam part of the foundation is rolled and bonded with screenings just as though it were a water-

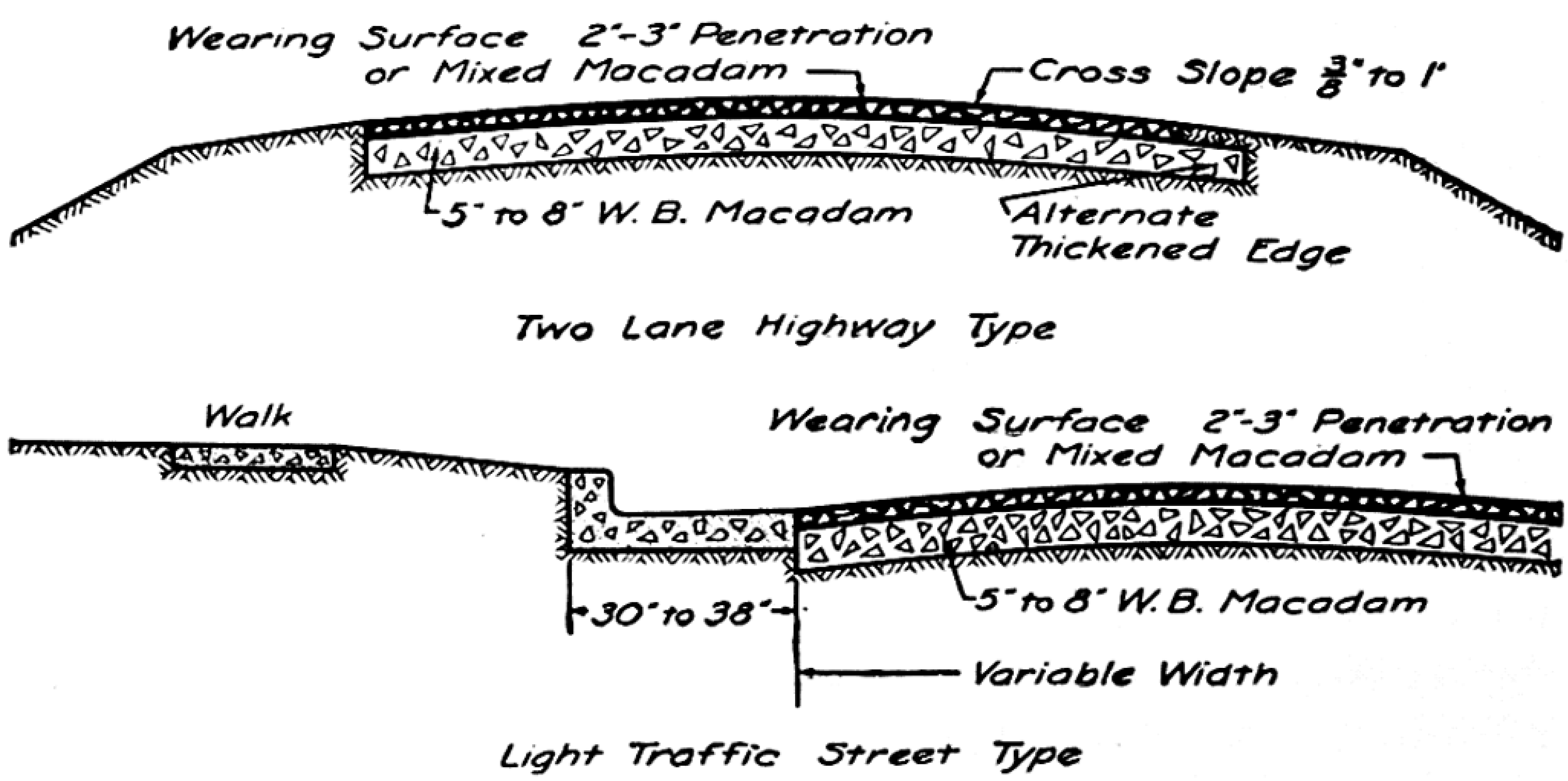


FIG. 97.—Typical cross-sections for penetration or mixed macadam roads and streets.

bound macadam. In many cases the penetration macadam is used for resurfacing old water-bound macadam roads that have become thoroughly compacted by traffic.

Sizes and Kinds of Stone.—Certain general principles governing the size of stone to use have been discussed in connection with water-bound macadam roads, and these apply to the base course for penetration macadam as well. The sizes commonly employed for the wearing surface are as follows:

A. Hard stone ranging in size from 1½ down to ½ in. Chips from the same kind of stone and ranging from ½ in. down but with the dust removed.

B. Medium stone ranging in size from 2½ down to 1 in. Chips screened through a ¾-in. screen and with the ¼ in. and finer removed.

C. Crusher-run stone of either hard or medium grade but containing no material passing a ¼-in. screen.

D. Stone of any one of the foregoing grades but screened to a fairly uniform size, such as $2\frac{1}{2}$ to $1\frac{1}{2}$ or 1 in.

E. When stone chips of suitable hardness cannot be obtained, pebbles screened from hard gravel may be substituted. The size ranges from $\frac{1}{2}$ down to $\frac{1}{8}$ in.

It will be apparent that when rolled, Class *A* stone will give a surface having smaller openings for the bituminous material to penetrate than will Class *B* stone and that Class *C* stone will produce a surface with smaller voids than either Class *A* or Class *B*. It will also be seen that Class *D* will produce a more porous surface than Class *A* or *B* or *C*.

It is therefore desirable to apply the bituminous binder to Class *C* stone after a light preliminary rolling and before any chips are spread. Class *A* stone is rolled thoroughly before the binder is applied, but no chips are spread before the binder is poured. Stone classed as *B* or *D* is rolled, and the surface voids partially filled with chips before any binder is poured. When Class *E* material is used it is substituted for chips and is handled in the same manner as chips. Obviously, none of these aggregates is well graded, and the void space in the material is high—probably in excess of 35 per cent. The wearing surface constructed by this method retains its shape under the kneading of wheels by virtue of the high internal friction obtained by rolling the angular stones until they are tightly packed, supplemented by the bonding action of the bituminous material.

Bituminous Binders.—Both tar and asphaltic binders are used for the penetration macadam, their general characteristics have been discussed, and typical specifications have been given in Chap. XIV.

Placing and Rolling Wearing-course Stone.—After the foundation course has been completed, the stone for the wearing course is spread to the required thickness. Whether or not it shall be rolled to final compaction before the bituminous material is poured into it depends upon the size of the stone and the texture of the surface. If the surface appears to be fairly well filled with fine material, as would be the case with Class *C* stone, it would be so dense when rolled that the bituminous binder could not penetrate it. Therefore, with such stone it is best to spread the bituminous binder after a preliminary light rolling. With other types of stone thorough rolling should precede the application of the bituminous material.

It is desirable to have as great mechanical stability in the wearing course as can be obtained, and therefore it is good practice to use a coarse stone like the types *A* and *B* and roll the layer thoroughly before the bituminous material is applied. If the stone has been rolled until the surface is closely knit together, there will be little danger of the finished road becoming rutted or uneven, and the bituminous binder will hold the chips in place and thus maintain a close-knit texture in the surface.

Examination of the surface after it has been rolled will show it to be made up of angular pieces of stone closely packed together, between which are voids of various sizes, depending upon the size of the stone and the thoroughness of the rolling. If the voids are large, they must be partially filled with chips to prevent the bituminous binder from penetrating too deeply into the layer. Usually with stone of classes *B* and *D* the application of chips at this stage of the construction is advisable, but with Class *A* stone the texture will be dense enough without the chips.

When chips are used they are thinly spread over the surface from piles alongside or from trucks equipped with spreaders and are brushed into the openings in the surface, any excess being brushed off at the edges. Care and persistence are necessary to secure an even texture in the surface, and this can be obtained only if the chips are carefully spread and properly brushed into it.

The chips used must be of tough stone; otherwise they will grind up rapidly under traffic because of their small size. If no hard, tough chips are available, clean, screened gravel of the proper size may be substituted.

Applying Bituminous Binder.—The bituminous binder is spread by a mechanical distributor (Fig. 98) which sprays the material on to the surface in the desired quantity and at the most favorable temperature in view of the atmospheric temperature and the nature of the material. Generally these materials are applied at a temperature of about 300°F., but the range is from 250 to 400°F. If the road has been properly prepared, the distributor, which it will be noted is quite heavy, can travel over the road without disturbing the stone enough to mar the smoothness of the surface.

The quantity of bituminous material required for the first application is from 1 to 1½ gal. per square yard of surface. The intention is to coat the surface completely and provide in addition enough binder to penetrate 1 to 2 in., thus affording a means of

cementing together the finer material in the mass comprising the upper layer of stone.

After the binder has been applied, the surface is dressed lightly with chips which are brushed into the voids with a slight excess

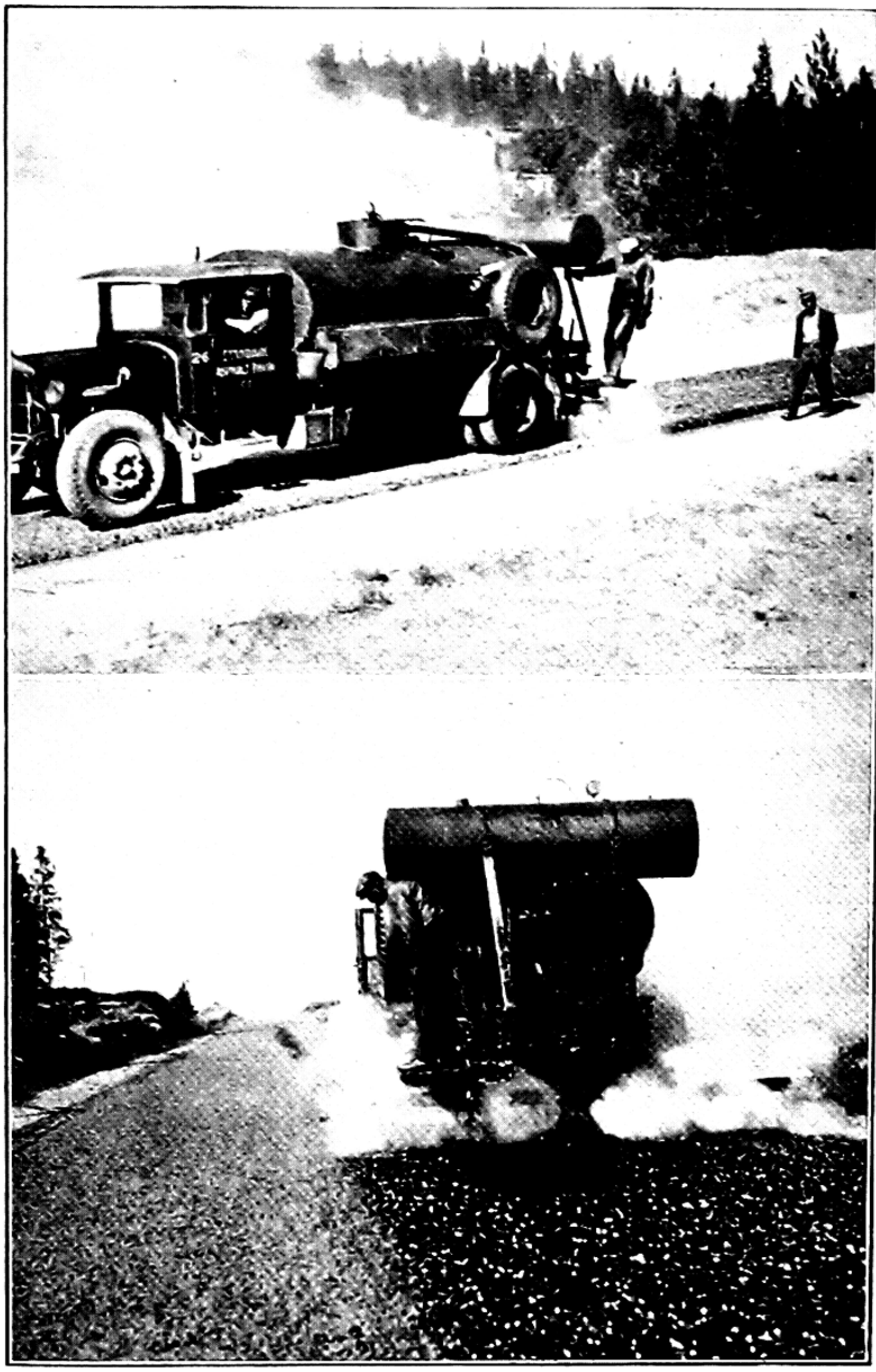


FIG. 98.—Spreading bituminous binder on penetration macadam.

on the surface. The road is then rolled a few times to smooth out the little unevennesses that develop during the process of applying the binder. The second application of bituminous material will consist of about ¾ gal. per square yard, spread in

the same manner as the first. This is immediately covered with chips and rolled. The surface is now ready for traffic if everything has gone well during the construction. Sometimes the texture of the finished surface is still too open to be waterproof, and a final spreading of $\frac{1}{4}$ to $\frac{1}{2}$ gal. of binder is needed to secure the desired density of surface. A final light dressing of chips, rolled into the surface, completes the construction of this seal coat.

Cost.—The cost of the penetration surface ranges from $8,000 to $12,000 per mile of two-lane surface, including both base and wearing surface. The wearing surface alone will cost from $5,000 to $7,500 per mile of two-lane road. These estimates of cost are general, and individual projects may cost a little less or considerably more than the foregoing suggestions.

BITUMINOUS CONCRETE

Several varieties of bituminous concrete have been developed in recent years to serve as dustless renewable wearing surfaces of high, antiskid properties. These varieties really fall into two clearly defined groups, although they appear under various names, and many minor differences in the methods of construction have grown up in various parts of the country. Even the nomenclature applicable to this group of surfaces differs materially between areas not so very widely separated. An attempt will be made herein to deal only with the basic principles of construction of each of the groups.

Bituminous Concrete Defined.—The bituminous concrete roadway surface consists of graded mineral aggregate and bituminous cement, the ingredients being proportioned to meet the requirements for stability under the conditions of service in the location. The materials may be mixed in place on the road by means of any one of several kinds of machinery, or they may be mixed in stationary mixing plants. The mixing may be performed at air temperature, or the materials may be heated before being mixed. The two classes differ not only in the grading of the aggregates but also in the construction methods employed. They are:

1. Mixed-in-place bituminous concrete, a type in which the void content of the aggregate ranges between 25 and 30 per cent.
2. Hot-mixed (plant-mixed) bituminous concrete, a type in which the void content is below 25 per cent and often below 20 per cent.

Bituminous Macadam Compared with Bituminous Concrete.—In the literature of the day the term "bituminous macadam" is employed to designate certain of the surfaces that are mentioned in the preceding paragraphs. Such a designation is perhaps valid for penetration macadam but is illogical for the other types. They are all bituminous concrete and differ one from another primarily in the grading (or lack of grading) of the aggregates. It will be convenient to think of these mixtures as consisting of ungraded aggregates, as is the case with penetration macadam and the thin mats; open-graded aggregates, with voids in excess of 25 per cent, as is the case with most of the mixed-in-place types and some of the plant-mixed types; and close-graded aggregates, with voids less than 25 per cent and often as low as 20 per cent, typical of the plant-mixed types. In each mixture there is provided a suitable asphalt cement.

Economic Status of Bituminous Concrete.—The bituminous concrete is looked upon as a type that can be adapted as a relatively low-cost renewable wearing surface capable of carrying a fairly large volume of traffic (up to 1,000 vehicles per lane per day), if the individual wheel loads exceeding 8,000 lb. are not too numerous and the climate not too severe, and in which the grading can be modified and the cost held relatively low for locations where the traffic is light, as, for example, on roads with an average traffic less than 500 per day. The surface will not long withstand traffic with tire chains, and the more open-graded mixtures do not resist long-continued wet weather too well, especially if there is alternate freezing and thawing.

When constructed with an asphaltic binder this type is called asphaltic concrete.

Conditions Required for Satisfactory Durability.—The bituminous concrete of the character built by experienced organizations under good specifications has proved its worth on a large mileage of highways. Throughout the region west of the Missouri River in the United States and in scattered areas in the remainder of the country, as well as in England and on the Continent, this type has been widely used and is standard construction for appropriate locations. Two conditions stand out as essential to success with this type, assuming as a matter of course that the surface will be well constructed.

1. There must be a thoroughly seasoned and stable foundation for the surface. Although the bituminous concrete type is widely used for resur-

facing roads of established stability, such as gravel or macadam, it is sometimes placed on newly constructed gravel or macadam, but this is questionable practice. It is preferable to use a thin bituminous mat on these newly constructed bases for a year or two until they are thoroughly seasoned and then apply the bituminous concrete surface.

2. The bituminous concrete requires skilled maintenance and should never be built on roads (county roads in some states, for example) that are not regularly maintained by a well-trained and properly equipped maintenance organization.

Foundations.—The asphaltic concrete wearing surfaces are extensively used for resurfacing macadam and gravel roads that have been in service a sufficient length of time to have become thoroughly compacted and to have demonstrated that they are sufficiently stable for the traffic. It is also widely used for resurfacing old concrete roads that have developed undesirable roughness.

Asphaltic concrete is also widely used for newly built foundations of gravel or macadam. These foundation courses are constructed of the thickness and grading required for stability under the traffic and climatic conditions prevailing in the region. Recently this type of wearing surface has been employed on foundation courses of densified, or stabilized, soils. Generally those foundation courses are at least 6 and often as much as 12 in. thick. It is the common practice first to finish the foundation with a thin mixed-in-place mat and permit traffic to use the road until the foundation is fully seasoned and of demonstrated stability.

Asphaltic concrete is often provided with a portland-cement concrete foundation, particularly in street-paving practice. In these types the design of the concrete foundations follows the principles discussed in Chap. XI.

MIXED-IN-PLACE BITUMINOUS CONCRETE

The mixed-in-place type of bituminous concrete (which is also widely known as bituminous macadam) consists of mineral aggregate and filler graded to the density deemed desirable for the service conditions to be met, mixed with bituminous cement on the road by means of blade graders such as are employed for general construction and maintenance work, tractor-drawn blade machines designed specifically for the purpose, or mixing machines of the pug type mounted to travel along the road. This

class of surface is also called "bituminous macadam," "road mix," "cold mix," and "oil mix."

Conditions Favorable to Mixed-in-place Type.—The mixed-in-place type of bituminous concrete has proved its worth on a large mileage of highways; but certain conditions, in addition to those common to all bituminous surfaces (page 389), are a prerequisite to satisfactory and economical service from this type. Although it has been constructed successfully with gravel aggregates, with broken stone or broken slag aggregates, occasionally with mixtures of gravel and broken stone, and with a variety of bituminous materials, failures have occurred when certain unfavorable conditions were encountered.

1. The aggregates must be air dry when mixed, and consequently this type is especially adapted to semiarid and arid regions. It may be constructed in the humid areas during the period of the year when dry weather predominates. If either the road surface upon which work is being done or the partially mixed materials are wetted by rains followed by hot weather, the materials will dry out sufficiently to permit the work to proceed. On the other hand, long-continued wet weather is likely to be disastrous to such a project.
2. Although the skill required for constructing this type is not so great as for the hot-mixed types (nor is the surface so durable), considerable experience on the part of the constructor and suitable equipment are necessary if the surface is to have good riding qualities and durability. It should never be attempted except under skilled supervision and with the facilities of a modern testing laboratory available to insure that the aggregate is properly graded and the correct amount of binder and filler is used.

Bituminous Materials for Mixed-in-place Surfaces.—The bituminous cement is incorporated in the mineral aggregate at air temperature by a process that requires the mixture to remain workable for several hours. Therefore the bituminous materials of the slow or medium curing types are used for aggregates graded to a low void content, and medium or rapid curing types for aggregates graded to a medium or high void content. The appropriate types of bituminous materials are indicated in Table XXVI. The M.C.2 is used when the mineral aggregate contains a good deal of fine material and the M.C.5 or R.C.5 when there is but little material of the size passing the 10-mesh sieve.

The mixed-in-place method of surfacing involves priming the surface of the macadam or gravel base upon which the mixing is to be performed. The priming material must have about the same quality as the dust layers; that is, it must be sufficiently

liquid to soak into the road and must be slow or medium curing so that it will *remain* long enough to penetrate. The materials listed in Table XXVI as S.C.2, M.C.1, and M.C.2 are used according to the atmospheric temperature expected in the period of the construction and the exact texture of the base course.

Mineral Aggregate for Mixed-in-place Surfaces.—The mineral aggregate for the mixed-in-place bituminous surfaces may be

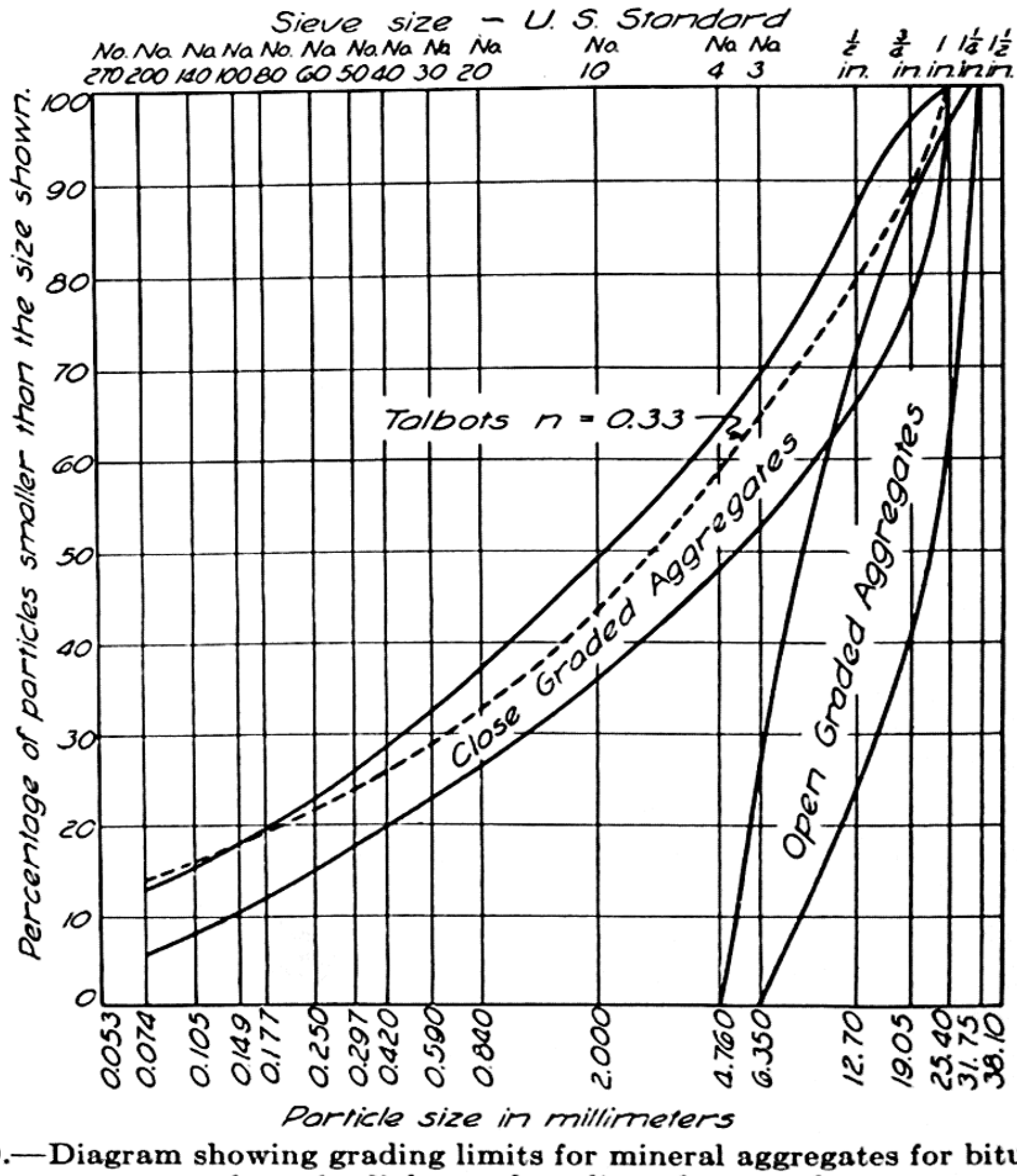


FIG. 99.—Diagram showing grading limits for mineral aggregates for bituminous surfaces for light- and medium-duty roads.

gravel, crushed stone, or crushed slag. These materials are classified as "close graded" or "open graded" according to particle-size distribution as shown by the sieve analysis and as a corollary to the void content of the mineral aggregate. However, this classification is an arbitrary one, and for each project the grading is determined by the traffic density and weight of loads expected on the highway and the materials available. In general, the trend is toward mixtures of a void content not over about

30 per cent. Where the stability must be a maximum, the close-graded type of aggregate is used with enough filler to reduce the void content to 25 per cent or less.

Close-graded Aggregate.—Close-graded aggregates are those that consist of particles of each size from coarse to fine in such proportions that the voids do not exceed about 30 per cent by volume and may be as low as 25 per cent (Figs. 99 and 100).[1]

A typical aggregate of this type is one in which all the material passes the 1-in. screen (called *minus* 1-in. stone and written "−1-in. stone"), 45 to 70 per cent passes the ¼-in. screen, and at least 5 per cent passes the 200-mesh sieve. These aggregates may consist of a mixture of crushed and uncrushed gravel, crushed stone, or crushed slag.

It seems desirable that gravel aggregates contain some angular material such as is obtained from crushing the oversize pebbles, but just how much of this is necessary as a minimum is a matter of debate. It appears that if at least 25 per cent of the aggregate consists of this angular material, there need be no apprehension about the behavior of the surface on the score of the mechanical stability of the aggregates.

Filler.—The close-graded mixtures contain a small percentage, usually about 5, of material that will pass the 200-mesh sieve. This should be looked upon as dust that serves to fill the small voids in the mixture. The filler is in part a product of crushing the stone and in part soil from the overburden of the gravel or rock deposit or from the shoulders along the roadway surface.

Open-graded Aggregate.—Crushed stone or crushed slag from which the finer portion has been screened is employed for mixed-in-place bituminous surfaces especially on old gravel or macadam where the traffic loads are not excessive and an open texture is desired for its antiskid properties. The sizes of the aggregates employed for this type are quite diverse, and it may be said that as a general rule the maximum size of the stone is not more than three-fourths the thickness of the wearing course layer. Such sizes as 1¼ to ½ in., 1 to ½ in., ¾ to ⅜ in. are typical for these aggregates; and materials of these and similar

[1] This method of presenting data on the grading of aggregates for bituminous surfaces was employed by Thos. E. Stanton and F. N. Hveem of the California Division of Highways in a report entitled "Control of Materials for Low Cost Bituminous Pavements," published in Part II of the *Fourteenth Annual Report*, Highway Research Board, 1935, p. 20.

sizes are known as open-graded aggregates. The void content of the aggregates will generally be in excess of 35 per cent by volume (see Fig. 99).

Quality of Aggregates.—The aggregates should be of such quality that the coefficient of wear is not less than 6 and pref-

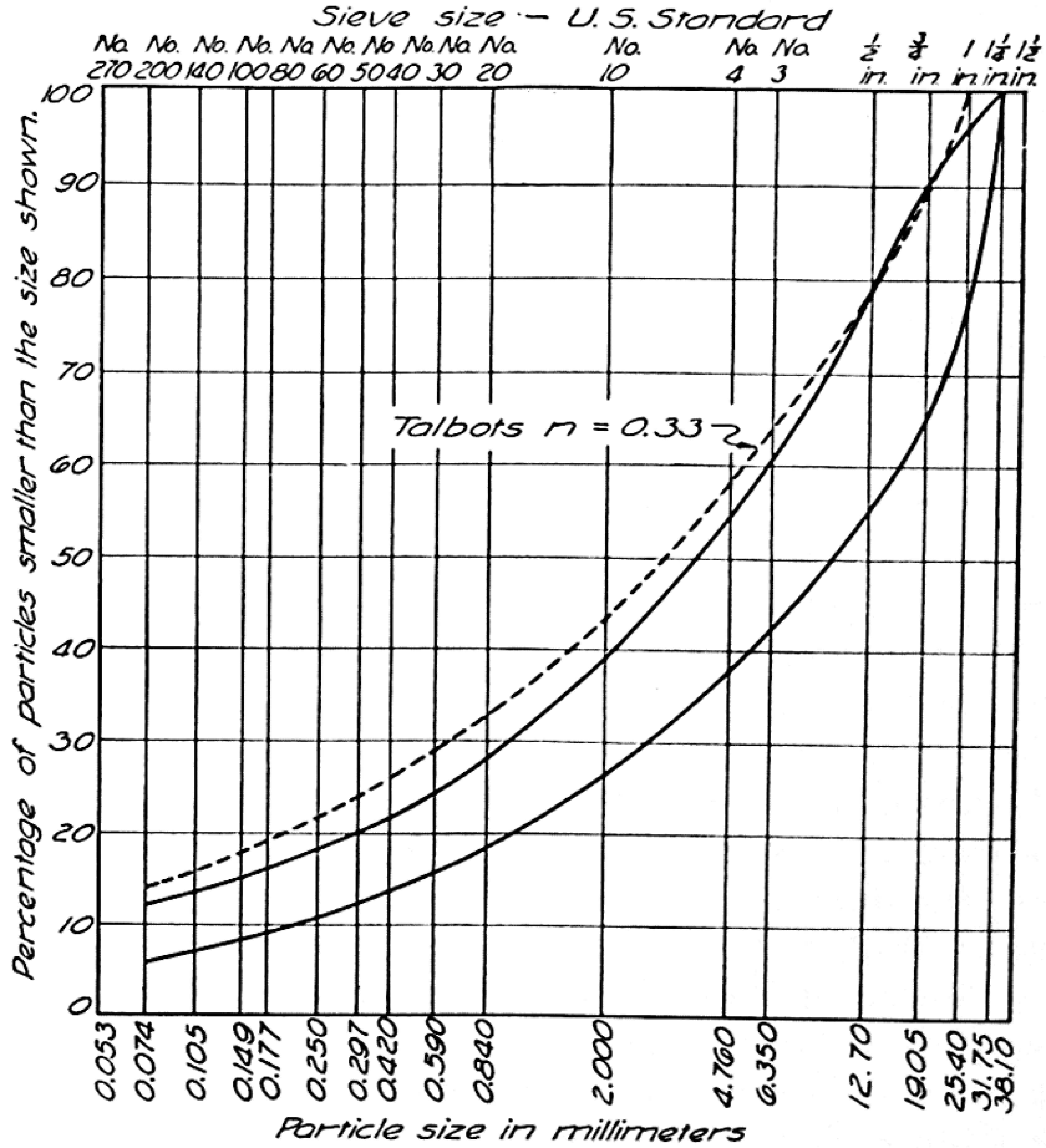


Fig. 100.—Diagram showing grading limits for mineral aggregates for asphaltic concrete on heavy-duty roads.

erably 8 according to the standard abrasion test,[1] or a maximum loss of 45 per cent for 500 revolutions in the Los Angeles rattler.[2]

The aggregates for these surfaces may be brought on to the job from an approved source of supply, in which case the quality and grading can be controlled as closely as desired; or they may be provided by scarifying the old road to a depth of 2 or 3 in. and the loose material thus provided be used as aggregate. The

[1] Test for Abrasion of Rock, A.S.T.M., D2-33, 1936 B.S., II, p. 1040.
[2] "Proposed Method of Test for Abrasion," *Proc. A.S.T.M.*, Vol. 35, Part I, p. 350, 1935.

material provided from the old surface can hardly be changed in grading very much except that material of the sizes needed to improve the grading can be added if that is deemed necessary. This method of construction is never undertaken unless the grading of the material in the old road surface is reasonably satisfactory.

Hydrophilic Aggregates.—The aggregates should be tested to determine their relative affinity for asphalt cement and water. No standard test has as yet been devised for this purpose, but a method whereby visual evidence may be obtained is as follows: A sample of the mineral aggregate is mixed with a sample of the bituminous cement in the specified proportions, cured, and then immersed in water. Visual inspection from time to time will indicate whether or not the water is separating the film of asphalt cement from the surface of the aggregate.

Swell Test.—The swell test is made to determine whether or not the fine mineral aggregate in a mixture is resistant to water. A sample of the surfacing mixture is made up, and the extent to which it swells when immersed in water is an indication of its stability on the road.[1] It is interesting to note that any swelling is an indication of unsatisfactory materials, although, if none better is available, aggregates that swell no more than ¼ in.

[1] This test was devised by J. W. Powers and is described in *Public Roads*, Vol. 11, No. 10, p. 192, December, 1930. It is as follows:

The machine used in the test consists of a cylindrical drum 28 in. in diameter and 20 in. in length, mounted longitudinally on a horizontal shaft and having a shelf 4 in. wide extending from end to end on the inside. The drum is charged with 14 cubical blocks of cast iron having rounded corners and edges and weighing a total of 5,000 g., along with 5,000 g. of rock, which is graded as follows:

Screen Size Inches	Total Percentage Passing
1½	100
1¼	80
1	60
¾	40
½	0

After charging, the drum is revolved 100 and 500 revolutions at a rate of between 30 and 33 r.p.m. The result is reported as the percentage of wear at 100 and 500 revolutions. At the present time the wear is considered that portion of the sample which, after test, will pass a 10-mesh sieve having a clear opening of 0.065 in. (No. 12 U.S. Standard).

by this test may be used with fair prospects for reasonable durability.

The material passing the 40-mesh sieve from close-graded aggregates should be tested for stability by being subjected to the standard tests for field moisture equivalent, which should not exceed 10 for construction in humid areas or 15 for dry areas; and for shrinkage, which should not exceed 3 per cent. These tests provide information to supplement that obtained by means of the swell test.

THIN MIXED-IN-PLACE SURFACES

Thin mixed-in-place surfaces are those ¾ in. or less in thickness and are employed on newly constructed gravel and macadams to serve until the road is well seasoned, after which additional thickness will be provided. These thin surfaces are also used to provide a smooth finish on bituminous or other pavements that have become uneven. Generally these thin types are made of open-graded aggregates ¾ to ⅜ in. in size and when completed and seasoned are substantially equivalent to the bituminous mats discussed on page 377.

This type of surface is used widely to provide a dustless wearing surface for a macadam or gravel base that has not yet become fully stable, as the first step in the construction of a thick bituminous concrete wearing surface. A subsequent application of another ¾ in. of wearing surface will provide a total of 1½ in. of bituminous top, which is adequate for most locations. This method of building up the bituminous top in successive layers is sometimes referred to as "stage construction" or "multiple-lift" construction. When constructed on a worn bituminous pavement it is called a "bituminous retread."

Priming Coat.—The first step in the construction of a thin mixed-in-place surface on a macadam or gravel base or as a means of improving an old gravel or macadam road is to clean the existing surface as described for the application of a road oil as a dust palliative (page 369).

The second step, when the foundation is gravel or macadam, is to apply a priming coat, usually of S.C.1 material, but macadams of open texture may be primed with M.C.1. This material is allowed several days to penetrate into the surface before the subsequent operations are started. If traffic cannot be kept off the treated surface, the operations are speeded up somewhat;

but by treating one-half the road at a time, traffic can be allowed to use the other half and will avoid the treated part fairly faithfully.

Binder Coat.—Frequently, bituminous surfaces, dense macadams, and hard pavements that are to be resurfaced with

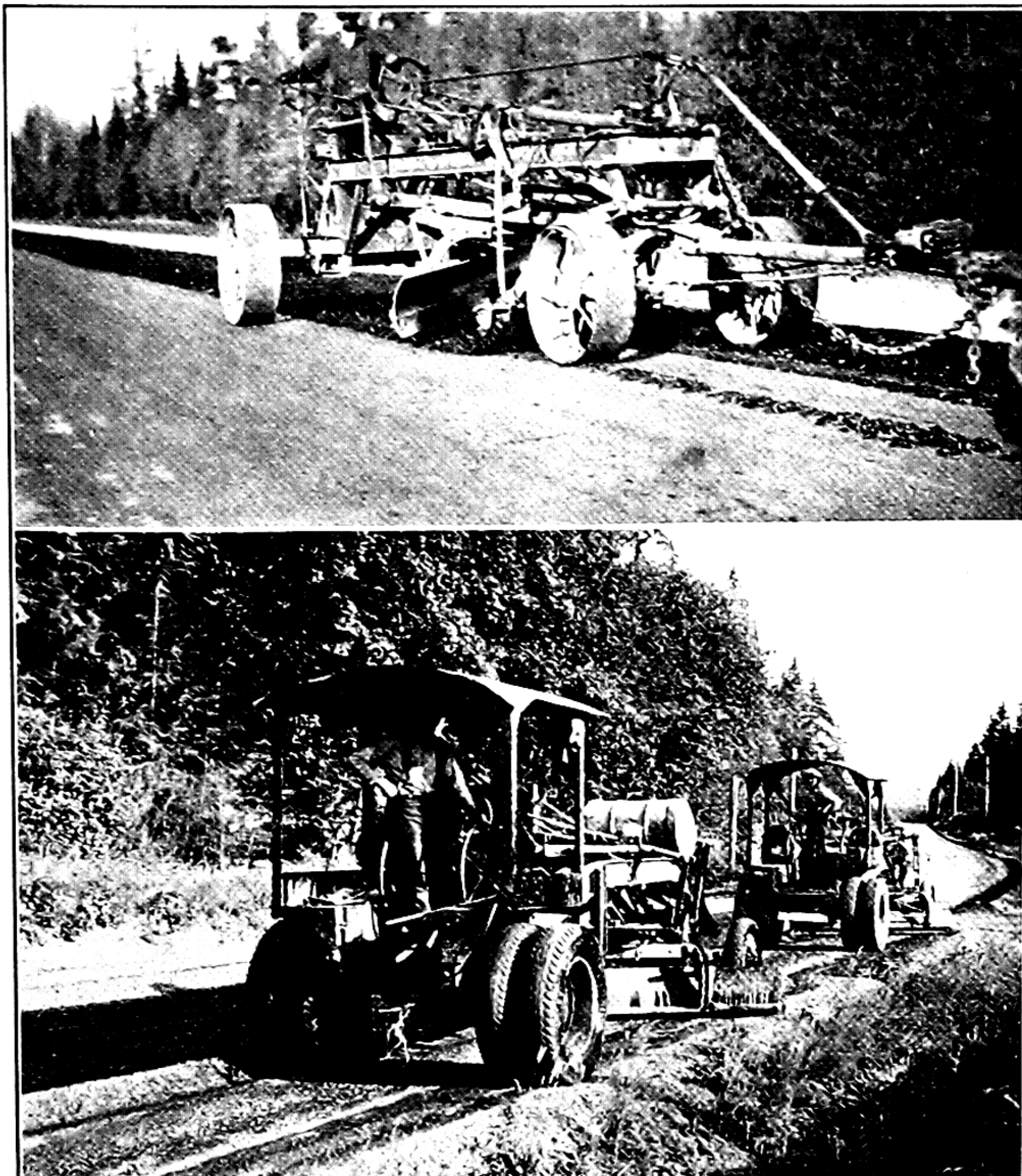


FIG. 101.—Blade grader mixing.

bituminous concrete need a binder coat (also called a "tack" coat) of bituminous material, for which purpose a cut-back of the R.C. series will be used, with R.C.4 or R.C.5 preferred if the weather is such that they can be handled. Sometimes the macadams will require a more fluid cut-back binder, such as

the R.C.2 or R.C.3, in order to secure some penetration into the surface. This type of binder course is sometimes used without the preliminary priming coat, but usually the priming coat is also required.

Mixed-in-place Course.—The mineral aggregate is spread over the surface that has received the priming coat, or binder, as may have been required, to the desired thickness and covered with the predetermined amount of bituminous material, usually about 3/4 gal. per square yard of R.C.3 or R.C.4, and then pushed into a windrow with a blade machine. It is then mixed by rolling the windrow from side to side with a blade machine until the stones are evenly coated with bitumen (Fig. 101). The layer is then spread and rolled.

Some engineers believe that it is desirable, before rolling, to cover the surface with a light dressing of crushed-stone screened to pass the 1/2-in. mesh (called "minus 1/2-in. stone," also "chips") with the dust removed. This dressing of crushed stone is sometimes referred to as an application of "key stone."

Seal Coat.—For best results the surface should be finished off with an application of about 1/4 gal. per square yard of a rapid-curing bituminous material, such as R.C.4 or R.C.5, which is covered with a light dressing of chips rolled into the surface.

THICK MIXED-IN-PLACE SURFACES

The normal mixed-in-place surface employed for locations where the traffic may reach 1,000 or more vehicles per lane per day is usually 1½ or 2 in. thick. It is made of aggregates graded to the density deemed necessary for the traffic. Consequently, the aggregates may be of the open-graded type, the close-graded type, or some intermediate grading.

Aggregates Obtained by Scarifying.—When the aggregates are to be obtained from the existing surface, it is scarified, and the loosened material scraped into a windrow on one-half of the base. If needed to correct the grading, additional material of suitable grading is added before the material is windrowed. The windrow is then struck off to a uniform cross-section by means of a notched template drawn along the windrow by a tractor. This assures the correct quantity of material for the thickness desired. The other half of the base is then primed with a bituminous binder, as already described, the windrow pushed on to the primed base, and the other half of the road primed.

From this stage the construction proceeds in the manner described on page 396.

Aggregates Hauled In.—When the surface is to be constructed of aggregates brought in instead of from material obtained by scarifying the surface, the aggregates from an approved source and of the desired grading are dumped on the primed base in a windrow which is struck off to a uniform cross-section by means of a notched template which leaves the correct quantity of material in the ridge.

Mixing.—The aggregate is spread over the surface in a uniform layer, and the bituminous material applied in the predetermined quantity (page 362). The mass is then once more scraped into a windrow which is rolled from side to side by the blade machines until the aggregates are thoroughly and uniformly coated with binder. The mixture is then spread and rolled with a tandem or three-wheeled roller weighing about 5 tons (Fig. 103).

It is the practice of some constructors to apply the bituminous material to the aggregates in the windrow instead of spreading the aggregates first. Probably the mixing from windrow requires a little longer, but the method is all right if it is desired to use it.

A traveling mixer is sometimes employed instead of the blade machines; and in that case the aggregates for one-half of the surface are placed in a windrow, and the bituminous binder applied. The mixing machine takes the material from the windrow, passes it through a pug-type mixer, and delivers the mixture to a spreader at the rear of the machine which spreads it over half the width of the normal two-lane road. The maximum width that these machines can spread is about 10 ft. The mixer has a piping system through which additional bituminous binder can be supplied if the stain test shows the mixture to be deficient in binder.

The adequacy of the mixture produced by any one of the foregoing methods is judged by the appearance of the stain test[1]

[1] The stain test is as follows: The sample from the roadway is first warmed. The fine material is then separated from the coarse by passing through a 10-mesh sieve. This can be readily accomplished by rubbing gently with the fingers and by loosening the fine particles that adhere to the coarse. The particles that do not pass the 10-mesh sieve may be discarded. The original sample should be of sufficient size to provide about 1 lb. of material passing the 10-mesh sieve. This 10-mesh sieve is heated to approximately

made in the field, which is checked from time to time by laboratory tests.

Seal Coat.—It is the general practice to finish these surfaces with a seal coat consisting of about ¼ gal. per square yard of bituminous material of the R.C.4 or R.C.5 type, which is covered with stone chips (key stone) which is rolled into the surface.

Costs.—Surfaces of the mixed-in-place type 1½ in. thick range in cost from $4,000 to $8,000 per mile of two-lane road, exclusive of any costs incurred in shaping or strengthening the old surface to convert it into an adequate foundation.

PLANT-MIXED ASPHALTIC CONCRETE

The asphaltic concrete that is mixed in a stationary plant is of the same general nature as the thick mixed-in-place surfaces, except that the grading can be controlled more adequately at a plant than on the road and the plant is designed for the hot-mixing method.

Nature of Plant-mixed Asphaltic Concrete.—Plant-mixed asphaltic concrete (which is also widely known as bituminous macadam) may be identical in composition with the mixed-in-place bituminous concrete except that it is frequently made with aggregates of a grading that must be more carefully controlled than is possible with the mixed-in-place process and is mixed in a stationary plant equipped to heat the ingredients and mix them while hot. The method is especially advantageous in areas where rainfall may be expected with some frequency

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the boiling point of water, which may be conveniently accomplished by placing the sample in a fruit jar or can and allowing it to remain partially submerged in the boiling water for a period of about 1 hr. The heated mixture is then dumped in a pile on the center of a sheet of white typewriter paper and leveled to a thickness of about 1 in., when another sheet of paper is placed on top. A wooden block 2 in. thick is placed on top of the paper, and to this are delivered five blows from a 2-lb. hammer, falling freely for a distance of about 1 ft. The two papers are then removed from the asphaltic mixtures, and the stain produced indicates the relative amount of oil in the sample. Satisfactory surface mixtures will produce a light yellowish-brown stain, in which the impression of the individual sand particles may be distinguished and which is not blurred or blotched. A heavy stain indicates the presence of excess oil, which is not only uneconomical but also causes displacement under traffic. It will be noted that these operations are of necessity carried out at air temperature and under conditions that preclude very accurate control of the mixtures. Nevertheless, many miles of durable surfaces have been constructed by this method. See also p. 422.

throughout the construction season, which would interfere with the operation of the mixed-in-place method. It is the only method feasible for street construction.

This type of surface is also called "plant mix" and when placed over a worn surface as a maintenance measure is called "retread."

A type of plant-mixed surface is used in some places under the name of "cold mix," which is an anomalous term, since both the binder and the aggregates are heated slightly before they are mixed.

Bituminous Material for Plant Mixes.—Since both the aggregates and the bituminous cement will be heated at the time of mixing, the bituminous cement may be of the medium- or slow-curing types. S.C.4 or M.C.4 are frequently employed. It is also possible to use in this process a straight asphalt cement (page 338) rather than a cut-back. In fact such material is preferable for work done in reasonably warm weather.

The standard hot-mixed asphaltic concrete is prepared at temperatures around 350°F., whereas the cold mix is handled at temperatures around 200 to 225°F. The cold-mix type requires a more liquid asphalt cement than the standard hot-mixed type, and materials like M.C.5 or S.C.5 would be used.

Aggregates for Plant Mixes.—The aggregates for the hot-mixed macadam may be gravel, provided at least 25 per cent of it is crushed material, crushed stone, or crushed slag. Although specifications vary considerably, a typical requirement for the grading of the mineral aggregate is given in Table XXVIII. But it must be recognized that all such grading specifications are general in character, and in practice the grading is adjusted within

TABLE XXVIII.—SIZE SPECIFICATION FOR AGGREGATES FOR CLOSE-GRADED ASPHALTIC CONCRETE

	PER CENT
Passing a 1¼-in. screen	100
Passing a 1¼-in. screen, retained on ¾-in. screen	20–30
Passing a ¾-in. screen, retained on ¼-in. screen	22–36
Passing a ¼-in. screen, retained on 10-mesh sieve	12–20
Passing a 10-mesh sieve, retained on 200-mesh sieve	16–26
Passing a 200-mesh sieve	2– 5

Specification for material passing the 10-mesh sieve

Passing a 10-mesh sieve, retained on 40-mesh sieve	30–50
Passing a 40-mesh sieve, retained on 80-mesh sieve	16–32
Passing an 80-mesh sieve, retained on 200-mesh sieve	16–32
Passing a 200-mesh sieve	8–16

Table XXIX.—Size Specifications for Aggregates for Open-graded Asphaltic Concrete

	Per Cent
Passing a $1\frac{1}{2}$-in. screen	100
Passing a 1-in. screen	90–100
Passing a $\frac{3}{4}$-in. screen	40– 75
Passing a $\frac{1}{2}$-in. screen	15– 35
Passing a $\frac{3}{8}$-in. screen	0– 15
Passing a $\frac{1}{4}$-in. screen	0– 5

these specifications to insure that the mineral aggregate has the grading and void content deemed to be required for the requisite stability of the service conditions to be expected.

The requirements as to the quality of the aggregates and filler are as given on page 394 for materials for mixed-in-place surfaces.

Mixing Plants.—The mixing plants are usually set up at a railway siding, because the bituminous materials are delivered in tank cars. Although the plants vary in details of design, they consist of the following essential parts:

1. A rotary dryer and heater for aggregates, usually operated with an oil-burning furnace.
2. A rotary screen to separate the heated aggregates into the desired sizes for recombining according to the specified grading.
3. A multiple-compartment storage bin for hot aggregates, with draw-off spouts to each compartment.
4. A steam-jacketed melting tank for the bituminous material with a steam-jacketed delivery pipe to the weighing bucket used in proportioning the asphalt.
5. A multiple-beam scale carrying a hopper into which the aggregates are drawn from the bins according to a weight proportion established for the project.
6. A twin-pug type of mixer with steam jacket and steam- or air-operated trap door.

The various makes of mixing plants are quite similar in their general make-up to the one illustrated in Fig. 102 but differ in the number of bins provided and consequently in the number of fractions into which the aggregates are separated for remixing in accordance with the prescribed proportions. The plants with two-compartment bins provide for two sizes of aggregates: for example, one size, that which will pass the $\frac{1}{4}$-in. screen; and the other, that which is retained on the $\frac{1}{4}$-in. screen and passes the 1-in. However, considerable variation in these sizes will be encountered in the practice in various parts of the country. The plant with a four-compartment bin is coming into rather general use because it permits more accurate proportioning than

smaller sizes, and particularly the filler, is a part of the routine of inspection and is handled in a laboratory set up at the mixing plant.

Preparation of the Surface.—Where a wearing surface of this type is to be laid on a gravel or macadam foundation a priming coat of bituminous material may or may not be applied to the existing surface; but it is good practice in most cases to apply such a priming coat, allowing it to stand uncovered for several days to make sure that the surface has absorbed all the bituminous primer that it can take. Excess primer has a tendency to fatten the mixture that is placed later, which is of course objectionable. Gravel roads of dense structure may require a binder coat (page 397) instead of a priming coat or in addition thereto. The macadams frequently require a binder coat. Newly constructed foundations of gravel or macadam are used for a time with only a thin mixed-in-place surface and then finished with a thick surface.

Placing the Surface Mixture.—The bituminous mixture is hauled to the project, usually in dump trucks, and is dumped into the hopper of a standard spreading machine (Fig. 103) and is spread on the surface of the road mechanically. The spreading machines propel themselves along the road and are provided with a power plant, a screed, and other equipment necessary for spreading the bituminous mixture to the desired thickness for the full width of the road or for half the width of three- or four-lane roads. These machines operate on side forms or on long-wheel-base skids and produce a surface that is exceedingly uniform in character. The spreading is followed by the customary rolling with the three-wheeled or tandem-type rollers, weighing about 5 tons. In many cases the surface will be placed in two layers, the lower one being of open-graded aggregates, and the upper one of close-graded aggregates.

The practice in some localities is to finish off the surface of these roads with a seal coat of bituminous cement and a dressing of stone chips (key stone). This final seal coat will be used when required to secure the close gritty texture often deemed the best from the standpoint of antiskid properties. The mechanical spreader for asphaltic concrete is not widely used on street work because of the variations in widths of streets and the lack of symmetry and uniformity in the shape of the pavement surface.

Doubtless it could be used much more widely than is now the practice if it were economical to do so.

Costs.—The surfaces of this type cost in the neighborhood of $6,000 to $8,000 per mile for a two-lane pavement and may be expected to serve a number of years with a minimum of maintenance. However, it is found that most of these surfaces eventually become somewhat uneven from frequent patching; and unless the patching is done very carefully, it will become necessary at some future time to add a second coating of about 1 in. of a similar mixture. This is sometimes called a "bituminous retread." The experience in many states has been that if this particular type of surface is properly laid, and if the maintenance is carried out in accordance with the best practice, the surface will maintain its smoothness and durability for many years. In fact, it doubtless can be maintained perpetually unless there grows up a class of traffic that includes individual loads in excess of the capacity of this type of road surface.

Ample evidence exists that in the long run these plant-mixed bituminous surfaces are the most economical of the bituminous types where the normal traffic exceeds 300 vehicles per lane per day and on week ends and holidays reaches the full traffic capacity of the road, provided the percentage of very heavy wheel loads is not too great, and where climatic conditions, especially alternate freezing and thawing, are not too severe.

Asphaltic Concrete for City Streets.—Asphaltic concrete of the close-graded aggregate type and a proprietary type known as *Warrenite-Bithulitic*, which is a close-graded aggregate type with a thin sheet-asphalt seal coat, are widely used for streets in cities and towns. The open-graded aggregate type of asphaltic concrete is used for streets in suburban areas. So far as the wearing surface is concerned, the theory of proportioning, the requirements for the aggregates, and the methods of placing are identical with the practice that has already been described for hot-mixed asphaltic concrete.

Foundation Course.—The common practice is to use a portland-cement concrete foundation for asphaltic concrete pavements on streets, although water-bound macadam and black base are each sometimes used in areas where suitable stone is available. The city street does not as a rule afford a good subgrade, and the difficulties in securing good drainage for the subgrade are much greater than on rural highways. There are

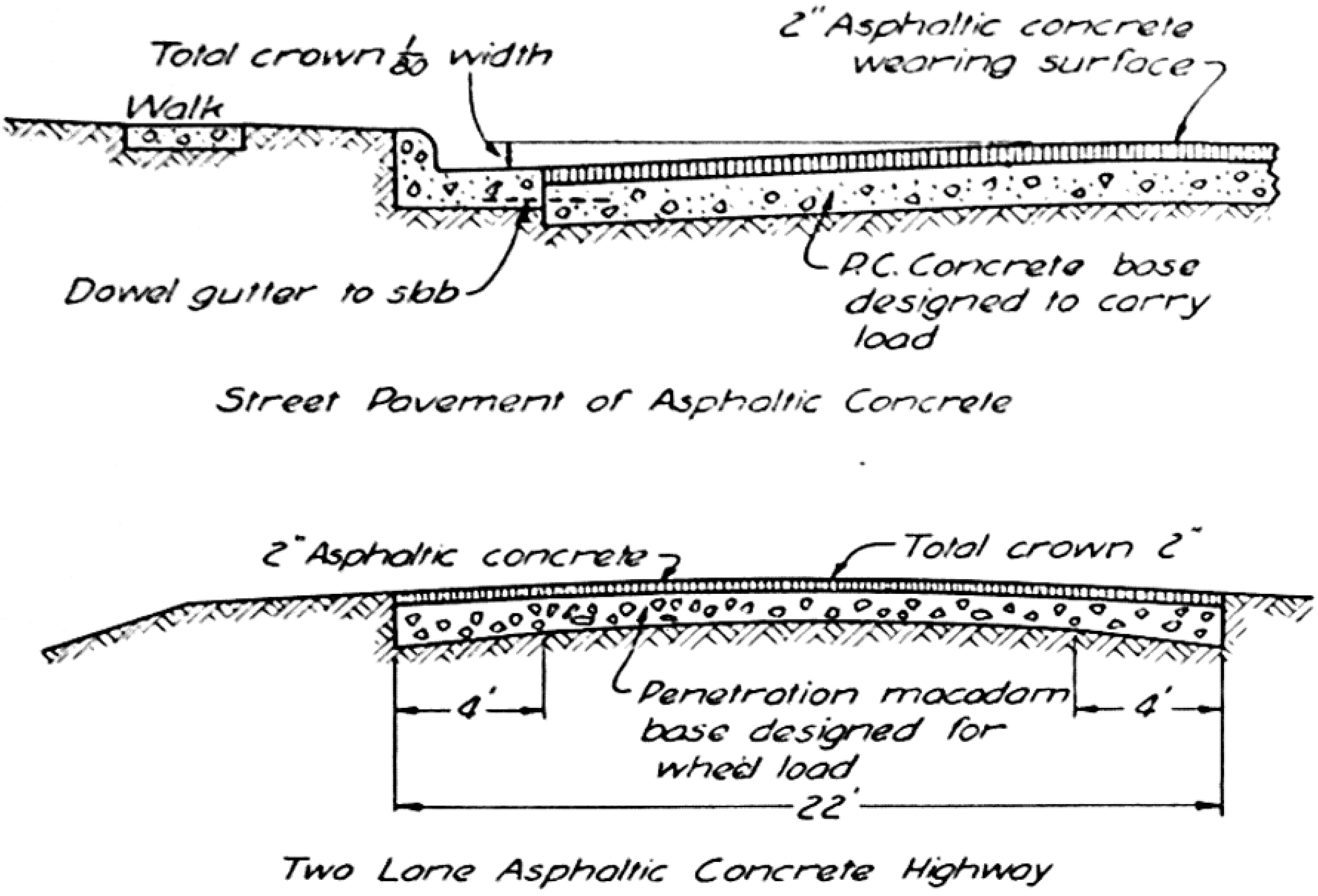


FIG. 104.—Cross-sections for asphaltic concrete surfaces.

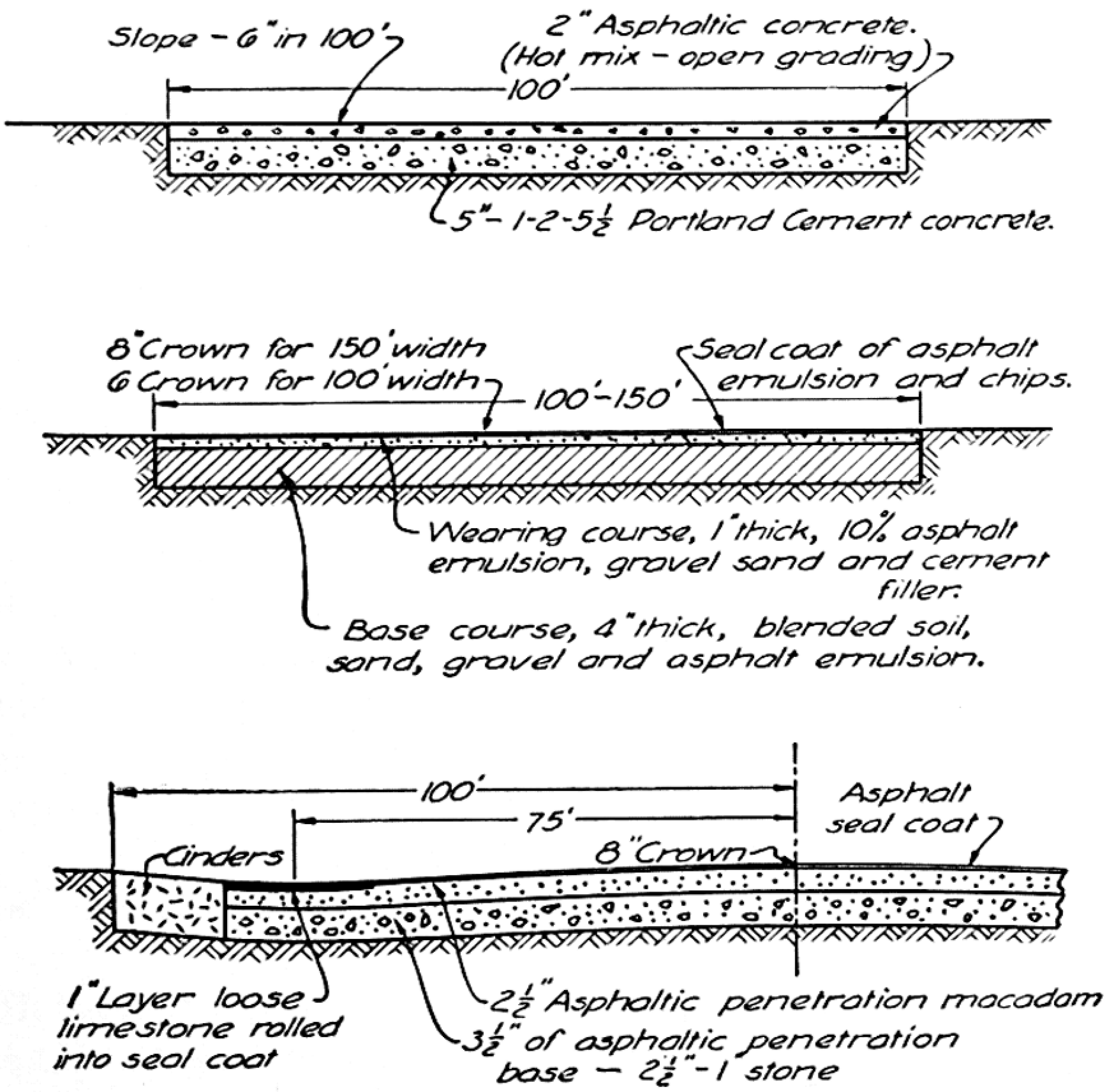


FIG. 105.—Cross-sections for airport runway surfaces.

numerous services under the pavement, such as telephone conduit, gas and water mains, sewer lines, and storm drains. In placing these, trenches have been dug and backfilled, with little or no tamping; and settlement is inevitable. For these reasons the concrete foundation for city pavements is usually preferred to the other types. The choice of material for the foundation is a matter of cost, and the type that will provide the requisite stability at the lowest cost should be selected. Cross-sections for asphaltic concrete pavements are shown in Fig. 104.

Table XXX.—Close-graded Asphaltic Concrete Surface Mixtures from Trunk Highways

Sieve sizes	Sample No. 1	2	3	4	5
	%	%	%	%	%
Bitumen	6.80	4.10	3.00	9.0	9.40
Passing 200-mesh	16.44	12.56	12.90	6.51	10.31
Passing 100, retained on 200-mesh	8.95	7.25	7.46	7.36	3.81
Passing 50, retained on 100-mesh	11.38	8.78	13.73	15.12	5.66
Passing 30, retained on 50-mesh	12.77	8.83	8.80	22.95	13.10
Passing 10, retained on 30-mesh	14.75	12.05	4.61	15.21	21.04
Passing 3, retained on 10-mesh	10.08	13.56	19.80	10.20	21.92
Passing 1 in., retained on 3	18.83	32.87	29.70	13.65	14.76
	100.00	100.00	100.00	100.00	100.00

The concrete foundation, or base course, is designed in accordance with the principles outlined in Chap. XI. In order to minimize the tendency for the wearing surface to creep on the concrete base, the base is finished with a sandy texture instead of with a trowel finish. Sometimes it is brushed to provide the slight roughness left by the broom marks. It has not been customary to provide a binder coat of asphalt cement on the base, although there is every reason to think that that might be well worth while.

Airport Runways.—Airport runways are usually about 100 ft. wide and from ¾ mile to more than a mile in length. The load to which they are subjected is that of landing planes; and in view of the size of the modern transport plane, the impact load at landing speed may reach a magnitude far in excess of the design

TABLE XXXI.—COMMERCIAL SIZES OF COARSE AGGREGATES[1]
(Crushed stone, gravel, and slag)

Size number	Nominal size square openings[2]	Amounts finer than each laboratory sieve (square openings), percentage by weight													
		3½ in.	3 in.	2½ in.	2 in.	1½ in.	1 in.	¾ in.	½ in.	⅜ in.	No. 4	No. 8	No. 16	No. 50	No. 100
1	3½ to 1½	90–100[3]	...	25– 60	......	0– 15	......	0– 15							
2	2½ to 1½	......	100	90–100	35– 70	0– 15	......	0– 5							
24	2½ to ¾	......	100	90–100	......	25– 60	......	0– 10	0– 5						
3	2 to 1	......	...	100	90–100	35– 70	0– 15	......	0– 5						
357	2 to No. 4	......	...	100	95–100	......	35– 70	......	10– 30	......	0– 5				
4	1½ to ¾	......	...	......	100	90–100	20– 55	0– 15	......	0– 5					
467	1½ to No. 4	......	...	......	100	95–100	......	35– 70	......	10– 30	0– 5				
5	1 to ⅜	......	...	......	......	100	90–100	40– 75	15– 35	0– 15	0– 5				
57	1 to No. 4	......	...	......	......	100	90–100	......	25– 60	......	0– 10	0– 5			
6	¾ to ⅜	......	...	......	......	......	100	90–100	20– 55	0– 15	0– 5				
67	¾ to No. 4	......	...	......	......	......	100	90–100	......	20– 55	0– 10	0– 5			
68	¾ to No. 8	......	...	......	......	......	100	90–100	......	30– 65	5– 25	0– 5			
7	½ to No. 4	......	...	......	......	......	......	100	90–100	40– 70	0– 15	0– 5			
79	½ to No. 8	......	...	......	......	......	......	100	90–100	40– 75	5– 25	0– 5			
8	⅜ to No. 8	......	...	......	......	......	......	......	100	85–100	10– 30	0–10			
9	No. 4 to No. 16	......	...	......	......	......	......	......	......	100	85–100	10–40	0–10		
10	No. 4 to 0[4]	......	...	......	......	......	......	......	......	100	85–100	.....	....	....	10–30
G1[5]	1½ to No. 50	......	...	......	......	100	80–100	......	50– 85	......	20– 40	15–35	5–25	0–10	0– 2
G2[5]	1½ to No. 8	......	...	......	......	100	65–100	......	35– 75	......	10– 35	0–10	0– 5		
G3[5]	1½ to No. 4	......	...	......	......	100	60– 95	......	25– 50	......	0– 15	0– 5			

[1] Promulgated by the Division of Simplified Practice, U.S. Bureau of Standards.
[2] In inches, except where otherwise indicated; numbered sieves are those of the U.S. Standard Size Sieve series.
[3] 100 per cent finer than 4 in.
[4] Screenings.
[5] The requirements for grading depend upon percentage of crushed particles in gravel. Size *G1* is for gravel containing 20 per cent or less of crushed particles; *G2* is for gravel containing more than 20 and not more than 40 per cent of crushed particles; *G3* is for gravel containing crushed particles in excess of 40 per cent.

TABLE XXXII.—TYPICAL USES FOR SIZES GIVEN IN TABLE XXXI

Use	Size number and nominal size[1]																			
	1	2	24	3	357	4	467	5	57	6	67	68	7	79	8	9	10	G1	G2	G3
	3½–1½	2½–1½	2½–¾	2–1	2–No. 4	1½–¾	1½–No. 4	1–⅜	1–No. 4	¾–⅜	¾–No. 4	¾–No. 8	½–No. 4	½–No. 8	⅜–No. 8	No. 4–No. 16	No. 4–0	1½–No. 50	1½–No. 8	1½–No. 4
Water-bound macadam:																				
Coarse aggregate	x	x	..	x																
Filler	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	x			
Bituminous macadam, penetration method:																				
Coarse aggregate	x	x	..	x																
Choke	..	..	..	..	..	..	..	x	..	x	x	..	..	..	x					
Seal	..	..	..	..	..	..	..	..	..	..	..	..	..	..	x	x				
Bituminous plant mixes[2]																				
Base or surface courses:																				
Base, open mix	..	..	..	x	..	x														
Base, closed mix	..	..	..	..	x	..	x	..	x											
Binder course	..	..	..	..	..	..	..	..	x	..	x									
Surface course, coarse grading	..	..	..	..	..	..	..	x	x	..	x									
Surface course, fine grading	..	..	..	..	..	..	..	..	..	..	..	..	..	..	x	x				
Seal	..	..	..	..	..	..	..	..	..	..	..	..	..	..	x	x				
Bituminous road mix:																				
Mixing course	..	..	..	..	..	..	..	x	..	..	x									
Choke	..	..	..	..	..	..	..	..	..	..	..	..	..	..	x	x				
Seal	..	..	..	..	..	..	..	..	..	..	..	..	..	..	x	x				
Drag leveling course:																				
Leveling course	..	..	..	..	..	..	..	..	..	..	x	x	..	x						
Seal	..	..	..	..	..	..	..	..	..	..	..	..	..	..	x	x				
Bituminous surface treatment	..	..	..	..	..	..	..	..	..	x	..	x	..	x	x	x				
Seal for airport construction	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	x				
Portland-cement concrete	x	x	..	x	x	x	x	..	x	..	x	..	x							
Railroad ballast:																				
Stone or slag	..	..	x	x	..	x	..	x												
Gravel	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	x	x	x
Roofing	..	..	..	..	..	..	..	..	..	..	..	..	..	..	x					

[1] In inches, except where otherwise indicated; numbered sieves are those of the U.S. Standard Sieve series.

[2] For plant mixes the aggregate should consist of appropriate sizes selected from Table XXX combined with suitably graded fine aggregate.

loads for highway pavements. Because of the high cost of the runways in any case, and in order to provide some cushion for landing planes, the flexible types of surfacing have been widely used for runways. The supporting foundation has usually been of gravel, broken stone, or soil and gravel composition, the materials being blended to secure the desired stability in accordance with the principles that have been discussed herein. The cross-sections[1] for several airport runways are shown in Fig. 105, and it should be noted that each element of the surfacing consists of an adaptation of one of the common types of roadway surfacing.

[1] Macatee, W. R., "Asphalt for Airports," Construction Ser. 45, The Asphalt Institute, January, 1939.

CHAPTER XVII

SHEET ASPHALT

The asphalt pavement in its modern form was first laid in Paris about the year 1858 and was followed by the laying of a pavement in London in 1869 and one in New York in 1870 or 1872. These pavements were constructed with a wearing surface of Val de Travers natural rock asphalt from Switzerland.[1]

The first asphalt pavement constructed with prepared mixtures was laid in Washington, D. C., in 1876.[2]

From these beginnings the asphalt pavement as known today was developed. By the application of the method of trial and error, mixtures were evolved to meet the needs of traffic until the heavy truck began to be the critical load. It was then learned that still further modifications in the mixture were needed to insure stability under the severest traffic, and laboratory research and field experimentation are being carried out to that end.

Meanwhile, ample technical information exists to enable the engineer to design asphalt pavements adequate for all ordinary traffic conditions.

The sheet-asphalt pavement consists of three layers or courses: a foundation course, a binder course, and a wearing course. The wearing course consists of a bituminous mortar composed of fine sand, finely ground mineral filler, and asphalt cement, and it is this layer that gives to the pavement its name, although it is not easy to explain how the name originated. A typical cross-section for a sheet-asphalt pavement is given in Fig. 106.

Design of Cross-section.—The foundation course gives the flexural strength to the asphalt pavement, and the first problem in the design of the cross-section is to design the foundation. The procedure outlined for the design of concrete-road slabs is followed if the concrete foundation is to be used.

The equivalent static load is determined as in previous cases, and no allowance is made for a reduction in the ordinary impact

[1] Abraham, Herbert, "Asphalts and Allied Substances," 2d ed., 1920, pp. 16, 17, 116, D. Van Nostrand Company, Inc.

[2] *Ibid.*, p. 16.

allowance because of the asphalt surface.[1] It is further assumed that the asphalt surface does not add to the flexural strength of the pavement.[2]

If the base course is of material other than concrete, the thickness adopted is based on the designer's estimate of the supporting power of the subgrade material and the probable stability of the base course that he proposes to use. Such foundation courses are principally of value in affording a good explanation for the failure of the pavement. They should be adopted only under conditions known, beyond doubt, to be favorable.

The sheet-asphalt surface is slightly plastic in cool weather and markedly so in hot weather. It must be adequately supported if it is to give normal service. There is some doubt whether the design of the sheet-asphalt pavement has kept pace with the growth of traffic loads, and many failures of this excellent type of surface have been attributed to defects in the wearing surface when they were obviously due to inadequate foundations.

Foundation.—To a very large extent the concrete foundation has superseded all other types for new sheet-asphalt pavements. It will doubtless become more and more the standard foundation for sheet pavements and, when properly designed and constructed, can be depended upon to support the wearing surface adequately.

Past practice has been to use concrete of such proportions as 1 part cement, 3 parts fine aggregate, and 6 parts coarse aggregate (1:3:6) or 1 part cement, 2½ parts fine aggregate, and 5 parts coarse aggregate (1:2½:5). As actually laid, these proportions produced concrete with a flexural modulus of rupture varying from about 100 to about 300 lb. per square inch.

At the risk of tiresome reiteration, attention is once more directed to the fact that the determination of the proper mixture for the concrete in the foundation course is a problem in design, the thickness of the foundation varying with the strength of the concrete. It must be attacked with a view to producing a concrete base of the proper strength at the lowest possible cost. The exact proportions will depend not only upon the strength

[1] TELLER, L. W., "Impact Tests on Concrete Pavement Slabs," *Public Roads*, Vol. 5, No. 2, p. 1, April, 1924.

[2] "Static Load Tests on Pavement Slabs," *Public Roads*, Vol. 5, No. 9, p. 1, November, 1924.

needed but also upon the relative cost of the constituent materials at a specific location.

The requirements for the quality of the concrete materials are those usually specified for high-grade concrete, save only that no great consideration is given to the wearing quality of the coarse aggregate.

The concrete foundation should be finished accurately to grade, being regularly tested with a straightedge and templates if need be, so that the asphalt top will be of reasonably uniform thickness. The base need not be float-finished, but it should have a surface free from protruding stone or areas deficient in mortar. A reasonably uniform mortar surface is the ideal to strive for. Artificial roughening is sometimes resorted to but is generally considered unnecessary.

Black Base.—There has been a relatively limited use of asphaltic concrete and asphaltic macadam for the foundation of the sheet-asphalt pavement, and a few examples of very good construction may be found where this type of base was employed. Probably the slowness with which this design has been accepted may be attributed in part to a lack of well-substantiated data upon which the base design may be predicated. Little enough is known with regard to the behavior of the concrete base, but even less is known as to the probable load-carrying capacity of the black base. As a consequence, precedent must be accepted as the chief guide in the design, extreme care being exercised to make sure that the loads and the subgrade soil are within the limits permissible for this type.

The thickness of the black base has varied in past construction from 2½ in. on some California work to 6 in. on some Middle Western jobs, but the general experience seems to show that 4 in. of good asphaltic concrete is needed on residence streets where favorable soil conditions exist and that 6 in. will be required for mixed traffic streets in the same location. Where the typical soils of the humid areas of the United States compose the subgrade, 6 in. would be about the minimum thickness to consider for the black base, even on light-traffic streets. The present state of knowledge of the strength of black base does not warrant using it for streets that carry loads in excess of about 3 tons.

The black base has its greatest economy in connection with the resurfacing of gravel or water-bound macadam where it is necessary to increase somewhat the strength of the existing surface in order to convert it into a satisfactory base before

placing the wearing surface. In many such cases 3 or 4 in. of the base mixture will suffice.

The binder course is omitted when sheet asphalt is placed on the asphaltic concrete base.

Water-bound Macadam Foundation.—Water-bound macadam has been used to a limited extent as the base for sheet asphalt, but it is much more common to use asphaltic concrete for the wearing course that is to be placed on macadam.

If constructed new for the purpose, the macadam base is designed and constructed in exactly the same manner as the wearing course of a water-bound macadam roadway surface. If

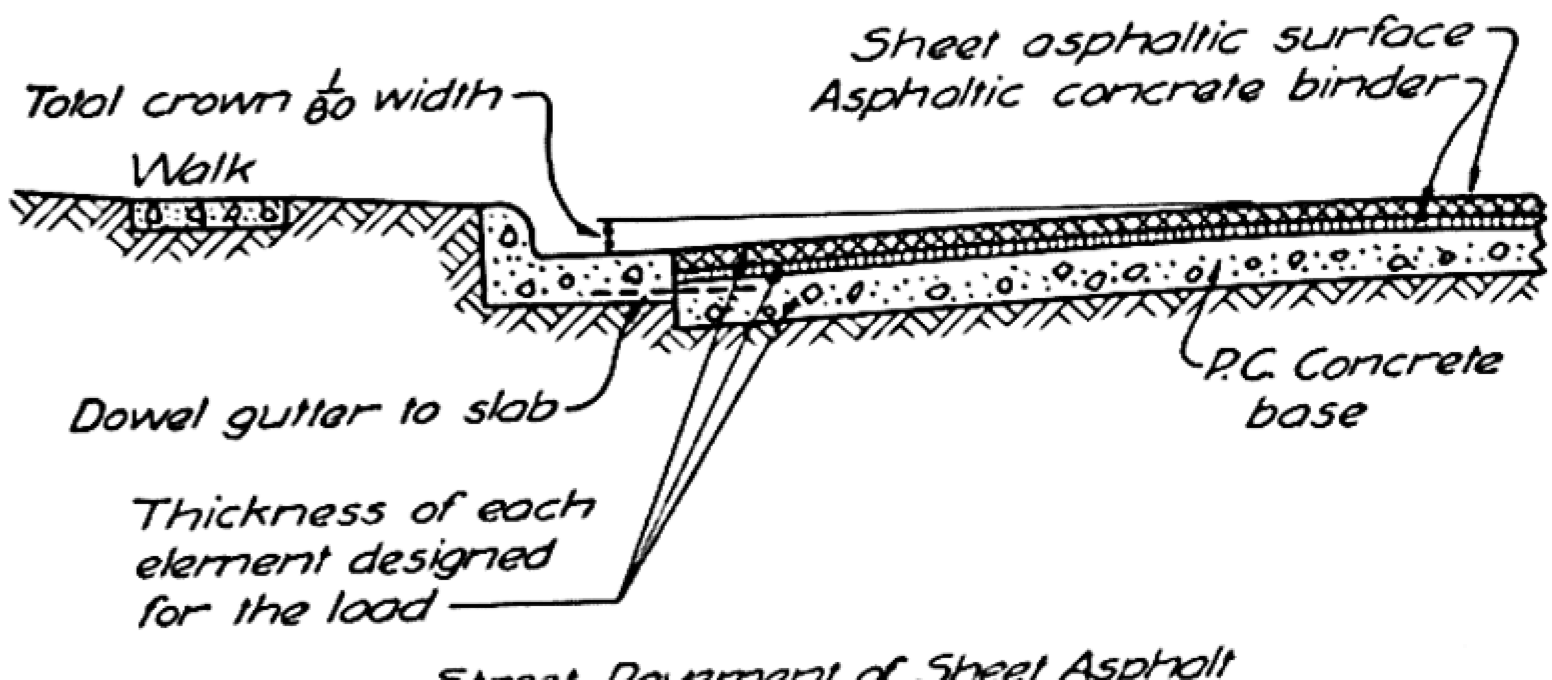


FIG. 106.—Typical cross-section for a sheet-asphalt pavement.

an old macadam road is utilized for base, it is repaired in the same manner as a macadam roadway surface that is being restored. Possibly a few inches of black base would be added to strengthen the macadam.

If the sheet asphalt is laid without a black base over the macadam, the usual binder course would be supplied.

Gravel Foundation.—Gravel is rarely used for the foundation for sheet asphalt except in connection with the black base and probably should never be used except in that way.

Worn Block Pavements for Foundations.—Sheet asphalt is admirably adapted for resurfacing brick, stone block, or concrete pavements that have worn until they are too rough for satisfactory service. The larger depressions in the old pavement are repaired with concrete, and the smaller ones filled with the binder mixture. The binder course is then laid, and the pavement is completed in the usual manner.

BINDER COURSE

Originally the sheet-asphalt wearing surface was placed directly on the base, but it was found to creep and become uneven, and

the intermediate, or binder, course was accordingly introduced. The binder course consists of a layer of asphaltic concrete placed directly on top of the concrete base. The thickness of the binder course depends upon the class of traffic and the total thickness of wearing surface that it necessitates. For heavy-traffic streets, the binder course is usually 1½ in. thick, whereas for residence or other light-traffic streets 1 in. is sufficient. For streets carrying traffic made up of very heavy individual loads, it is recommended that the binder be 2 in. thick.

The binder mixture is of two types, known respectively as open binder and close binder, the difference being in the grading of the mineral aggregates.

Open Binder.—The open binder is composed of stone ranging in size from ¼ or ½ to 1 in., to which is added from 5 to 8 per cent of asphalt cement. If the stone does not carry some fine material such as will pass the No. 4 sieve, from 10 to 15 per cent of ordinary concrete sand may be added. Other size limits for stone are sometimes specified, but in this type of binder no attempt is made to secure a dense mixture, nor are the voids filled with the asphalt cement, as is apparent from the proportions given. The asphalt cement must be sufficient in quantity to coat all the stones so that when rolled the binder will be well cemented together. The surface of the binder course after it is rolled will be porous and open; hence the designation that is given it. The surface interstices of the binder course will be filled with the surface by the rolling.

Close Binder.—Close binder differs from open binder in that the mineral aggregate is graded so as to secure a fairly dense mixture in which the large voids are fairly well filled. This is accomplished by mixing broken stone and sand in such proportions that the sand will fill the voids in the stone. The stone ranges in size from 1 in. down, and crusher-run is frequently employed. The sand should be well graded, but the requirements are not so rigid as those for sheet-asphalt surface mixtures.

The amount of asphalt cement required for the close binder is somewhat greater than that required for the open binder.

The close binder is recommended for streets carrying moderate or heavy traffic, and the open binder is permissible only on very light-traffic residence streets.

Character of Materials for Binder Course.—The stone for the binder course must transmit the load from the wearing surface to

the concrete base; but since the medium is somewhat plastic, the requirements for the stone are not exceedingly rigid. Any hard tough limestone, trap rock, or granite may be used. The stone should be clean and should run with reasonable uniformity as to grading. The sand for the close binder should be of the quality and cleanness required for high-class concrete. Gravel pebbles are sometimes used for coarse aggregate in close binder.

Old Surface Mixtures for Binder.—Old sheet-asphalt surfaces that have been removed in resurfacing streets are occasionally utilized for the preparation of close binder. The old surface mixture is melted in kettles heated by steam and then mixed with the proper amount of stone. Additional asphalt cement is added to renew the life of that which is in the old surface mixture. Suitable precautions must be taken to insure that the old mixture is thoroughly broken up and distributed through the mass of stone and that the right amount of new asphalt cement is added. The asphalt cement in the ordinary binder course is of the same quality as that used for the surface mixture except that it can advantageously be about 10 points softer.

SURFACE MIXTURE

The surface mixture for the sheet-asphalt pavement consists of sand, filler, and asphalt cement.

Sand.—It has long been recognized that the stability of the sheet-asphalt surface depends to a large extent upon the gradation of the mineral aggregate. Experience with cement mortars has shown the importance of using a sand that is properly graded, to insure density and strength, and the sand grading is no less important in the sheet-asphalt surface mixture. Inasmuch as the mineral aggregate comprises about 90 per cent of the volume of the surface, it is reasonable to expect that it needs to be carefully selected and properly graded. The sand should be clean and free of slate, shale, or other friable material. It is generally thought that sharp sand is better than sand made up of rounded particles, but this cannot be stated as a general fact because successes and failures have been recorded with both classes of sand.

It does appear to be established that asphalt cement adheres more readily to some types of sands, particularly those which have grains with a rough surface, than to those with polished grains. Laboratory tests will indicate which sands are preferable and will

also serve to check the stability of various mixtures of sand and filler. The laboratory should be utilized more generally for this sort of study than seems to be the practice at present.

Filler.—The filler is the portion of the mineral aggregate passing the 200-mesh screen and may consist of finely ground rock dust or of portland cement. The filler plays an important part in the mixture, since it serves to fill the small voids and to give density to the surface. A good filler should not only be fine enough to pass the 200-mesh sieve but also should contain a good percentage of particles that will pass a 400- and also a 600-mesh sieve. Since sieves are not made commercially with more than 200 meshes per inch, the amount of the finer particles is estimated by a modification of the method of elutriation which is employed in soil analysis. Portland cement is a desirable filler, since it is finely ground and is made up of very durable particles. It is more expensive than limestone dust, and for that reason the stone dust is often used. Limestone dust ground to the proper degree of fineness is widely used for filler, and although it is possibly less stable than portland cement, it is satisfactory. Silica dust is also employed for filler and seems to be as good as portland cement. In the preparation of stone dust for the filler the stone is first heated so as to dry it thoroughly and then crushed and finally ground in a ball mill using flint pebbles for the abrasive. Some grinding mills use steel rollers for grinding, but the resulting product is usually not so finely ground as that produced in the ball mill. The filler should be ground to such a fineness that at least 70 per cent will pass the 200-mesh sieve and 95 per cent the 100-mesh sieve.

Theory of Proportioning.—Most of the sheet-asphalt gradings that have been used in the past were developed by trial, and the mixtures employed in any city were usually adopted after the various sands available to contractors at that place had been examined and perhaps used on a few contracts. The proportioning was largely a matter of hit or miss, no theoretical basis having been established.

In recent years some important researches[1] have been conducted with a view to establishing a correct basis of universal application for porportioning asphalt mixtures, and considerable progress has been made in that direction.

[1] Hubbard, Prévost, and F. C. Field, "Researches on Asphalt Pavements," *Circ.* 34, Asphalt Assoc., New York, N. Y., 1926.

At the present time there are really two schools of thought with reference to proportioning; one is that the mineral aggregate should be so graded that a low percentage of voids will be obtained (a theory that was current 20 years ago and later abandoned) and that the asphalt cement serves to fill the voids; consequently, the quantity of asphalt cement to use will depend primarily upon the void content of the mineral aggregate.[1] The other theory is that the asphalt cement serves to coat the particles of the mineral aggregate and, therefore, the quantity of asphalt cement required will vary with the total surface area of the aggregates. According to this idea, the greater the quantity

Fig. 107.—Spreading sheet-asphalt mixture.

of fine material in the mixture the greater the quantity of asphalt cement that will be required, which checks out only fairly well in practice.

It is impossible at this time to produce conclusive evidence in support of either theory, although there are strong indications that the void theory possesses considerable merit if the correct interpretation of the void test can be found.

Meanwhile, it is necessary for the engineer to rely to a considerable extent upon the old empirical standard gradings as a basis for proportioning asphalt mixtures. This he may do with considerable assurance, as is attested by the enormous yardage of good sheet asphalt laid each year.

[1] Skidmore, Hugh W., "Sheet Asphalt Mixture Research," *Eng. and Contracting* (*Roads and Streets*), Vol. 63, No. 4, p. 693, April, 1925.

A mixture of sand and filler may be designed to give a low percentage of voids, but whether the mixture as laid actually has a low percentage of voids will depend upon whether it is a workable mix. If it is too tough and dry the rakers will not be able to spread it quickly and consequently will not comb it thoroughly; and if they fail to do so, the mixture will not be uniform in texture or thickness after rolling. The hot asphalt cement serves as a lubricant to the particles of sand during the rolling, and dry harsh mixtures will not compact to the theoretical density because of lack of mobility. The standard gradings afford a good starting place in the design of mixtures and were the basis of most of the gradings in use in 1928.[1]

TABLE XXXIII.—STANDARD SAND GRADINGS

Sieves	Heavy traffic, per cent		Light traffic, per cent	
Passing 10-mesh, retained on 20-mesh	5		10	
Passing 20-mesh, retained on 30-mesh	8	23	10	35
Passing 30-mesh, retained on 40-mesh	10		15	
Passing 40-mesh, retained on 50-mesh	13	43	15	45
Passing 50-mesh, retained on 80-mesh	30		30	
Passing 80-mesh, retained on 100-mesh	17		10	
Passing 100-mesh, retained on 200-mesh	17	34	10	20
Passing 200-mesh	0		0	
	100		100	

SPECIFICATIONS FOR SAND GRADING

Passing 10-mesh	....	100
Total passing 10-mesh, retained on 40-mesh	....	12–50
Passing 10-mesh, retained on 20-mesh	2–15	
Passing 20-mesh, retained on 30-mesh	5–15	
Passing 30-mesh, retained on 40-mesh	5–25	
Passing 40-mesh, retained on 50-mesh	5–30	
Passing 50-mesh, retained on 80-mesh	5–40	
Total passing 80-mesh, retained on 200-mesh	....	20–40
Passing 80-mesh, retained on 10-mesh	6–20	
Passing 100–mesh, retained on 200–mesh	10–25	
Passing 200-mesh	0–5	

[1] "Sheet Asphalt," Brochure 9, 3d ed., The Asphalt Association, New York, N. Y.

SMITH, FRANCIS P., "Construction of Hot Mix Pavements," *Can. Eng.*, Vol. 32, No. 2. p. 132, Jan. 11, 1927.

Established Gradings.—The sand gradings shown in Table XXXIII are widely used for sheet-asphalt surface mixtures. They have been well established in practice and the nearer the surface mixture approaches these in composition, the more likely the pavement will be to having the desired stability. It will be noted that the various sizes are separated into three groups, and while it is desirable to have the specified amount of each size, it is not always possible to secure sands so graded. If each group is present in the right amount, that is usually the best that can be accomplished. These gradings must be relied upon unless laboratory studies of the materials to be used indicate desirable modifications.

The finer portion of the mixture is the most important, and the grading may more safely depart from the standard as regards the material above the 40-mesh size than as regards the material finer than the 40-mesh. The amount of filler that can be used in a mixture depends upon the amount of fine sand present. The trend of present-day practice is toward the use of 18 per cent or more of filler. An analysis of a sheet-asphalt mixture that has successfully withstood heavy traffic for a number of years is given in Table XXXIV. This mixture is typical of good practice.

Table XXXIV.—Analysis of a Typical Asphalt Mixture[1]

Bitumen	10.6	
Passing 200-mesh sieve	18.3	
Passing 100-mesh, retained on 200-mesh	16.5	45.5
Passing 80-mesh, retained on 100-mesh	10.7	
Passing 50-mesh, retained on 80-mesh	22.8	30.2
Passing 40-mesh, retained on 50-mesh	7.4	
Passing 30-mesh, retained on 40-mesh	3.6	
Passing 10-mesh, retained on 30-mesh	9.3	13.7
Retained on 10-mesh sieve	0.8	
	100.0	

[1] From the author's files.

Asphalt Cement Required.—The amount of asphalt cement required for the mixtures provided for in Table XXXIII will vary between 9 and 13 per cent of the total mixture including the asphalt. There is no basis other than the pat test and visual inspection based on long experience for determining the exact amount of asphalt to use. In the laboratory, preliminary tests may serve to indicate the approximate amount of asphalt cement required, but the mixtures produced at the asphalt plant will

seldom be exactly like the laboratory samples. Hence, the quantity of asphalt cement must be determined for the run of the plant on the basis of the pat test, supplemented by stability tests on samples taken from the pavement after the mixture has been rolled. Fortunately, small variations in the quantity of asphalt show up plainly in the appearance of the mixture and in the stain produced by the pat test.

The Pat Test.—A small wooden paddle with a blade 3 or 4 in. wide, 5 or 6 in. long, and ½ in. thick, tapered to an edge at one end and with a convenient handle at the other, is used to take as much of the hot mixture from the wagon as it will hold, being careful to avoid any of the last droppings from the mixer which may not be entirely representative of the average mixture. Samples of mixture should never be taken from the mixer itself but only from the wagon after mixing is completed.

In the meantime a piece of brown Manila paper with a fairly smooth surface, 10 to 12 in. wide, and torn off at the same length from a roll of this paper, which can be had at any paper warehouse, is creased down the middle and opened out on some very firm and smooth surface of wood—not stone or metal, as this would conduct the heat too rapidly. The hot mixture is dropped into the paper sideways from the paddle, and half of the paper is doubled over on it. The mixture is then pressed down flat with a block of wood of convenient size until the pat is about ½ in. thick.

The paper will be found to be stained to a different degree depending upon whether there is a deficiency, a proper amount, or an excess present.

In this the amount of asphalt cement to use in making a mixture can readily be regulated, and the pat papers obtained will be evidence of the character of the mixture turned out. Where a laboratory examination is to be undertaken, a sample of surface mixture which is made from the material compressed between the paper can be used for this purpose, trimming it down into the proper form and sending it, accompanied by the paper, for the purpose.

Proportioning Asphalt Cement.—The proportion of asphalt cement is determined by weight. Since it is desired to use enough bitumen to coat all the particles, it is apparent that a more rational method would be to proportion the asphalt cement by volume. As a matter of convenience and accuracy, however,

weighing is resorted to. The weight of asphalt cement per unit of volume varies considerably; and if the proportioning is by weight, account must be taken of the variation in the specific gravities of the asphalt cements that might be used. Attention is also called to the fact that the percentage of bitumen is always specified, not the percentage of asphaltic cement. This is necessary because of the differences in the proportion of bitumen in the various asphalt cements. In determining the amount of asphalt cement to use, it is therefore necessary to take account of the specific gravity and bitumen content of the cement.

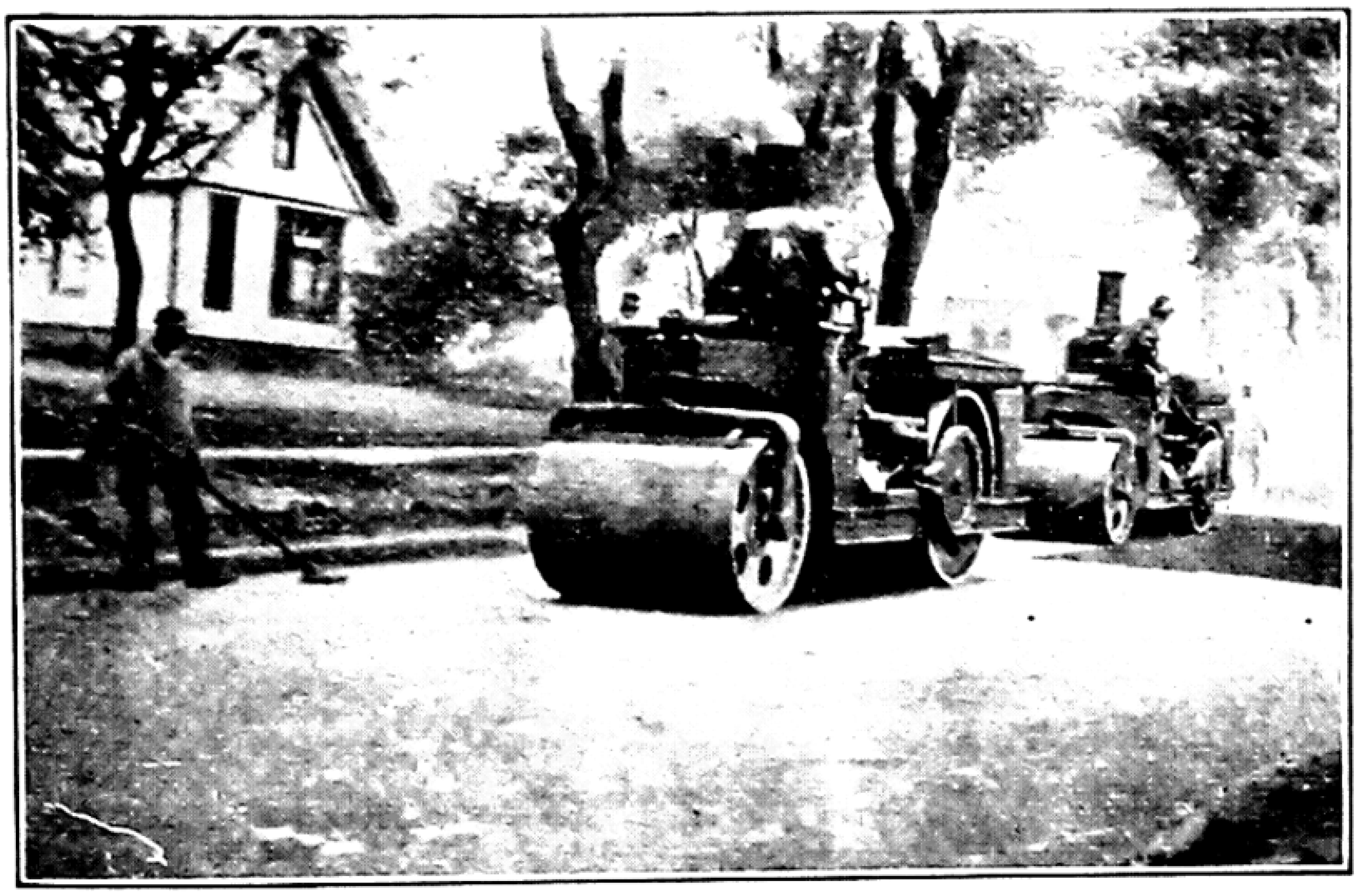


FIG. 108.—Rolling sheet-asphalt surface.

To illustrate the significance of these facts in proportioning the asphalt cement in a paving mixture, consider a sheet-asphalt surface mixture in which the percentage of bitumen by weight is 8.9. Assume that Trinidad Lake asphalt cement was used in which the percentage of bitumen is 65 and the specific gravity 1.28. The percentage by weight of asphalt cement necessary to give 8.9 per cent of bitumen is $8.9/0.65 = 13.5$ per cent.

If, however, an asphalt cement produced from petroleum were used, the percentage of bitumen in the asphalt cement would be about 99.5, and the specific gravity about 1.03. The percentage of the asphalt cement required by weight would be substantially the same as the percentage of bitumen.

Since the bitumens extracted from the various asphalt cements vary somewhat in specific gravity, a proportion based on weight is not exactly a constant proportion by volume. This is the

reason for the variation in the allowable percentage of bitumen provided for in the specifications.

CONSTRUCTION OF THE PAVEMENT

The concrete base is finished, and ample time is allowed for the concrete to set before the binder course is placed. If the binder is hauled too soon, injury to the base will result because of the weight of the trucks used for hauling. The base should be clean and dry when the binder is spread.

The binder course is mixed in the twin-pug type of mixer which is also used for mixing asphaltic concrete. The stone must not be too hot, or some of the asphalt cement will run off while the mixture is being hauled; it must not be too cold, or it cannot be spread and rolled satisfactorily. For most materials 275 to 325°F. is about the permissible range of temperature, but this is subject to some variation for the different asphalt cements.

The binder is dumped on the concrete in advance of the place where it is to be spread and is shoveled into place and raked to a thickness that will, when rolled, give the prescribed thickness of binder course. Experienced rakers need no guide in this operation, and usually no guide is employed except possibly a line on the curb at the top of the layer. A check on the average thickness is obtained by computing the area that each load should cover to the prescribed thickness. Since the mixture is proportioned by weight, the weight per load is known.

As soon as spread, the binder is rolled, first with a 3-ton tandem roller and later by one weighing 8 to 10 tons. The rolling must proceed rapidly so that compression is secured before the asphalt cement cools too much. The rolling is carried out both crosswise and longitudinally and is a vital part of the construction, especially as regards the evenness of the finished surface. Only a skilled operator can be expected to secure satisfactory results.

The wearing course must be placed soon after the binder course has been rolled. It is especially desirable that close binder be covered with the wearing course the same day it has been laid, and it is safer to follow the same rule for all classes of sheet-asphalt construction.

The mixture for the surface is prepared by weighing the sand, filler, and asphalt cement into a twin-pug mixer and thoroughly combining the ingredients by mixing. The temperature should be somewhat higher than for the binder material. With many

kinds of asphalt cements a temperature of 350°F. may be reached without injury to the materials. Other asphalts must be handled at a somewhat lower temperature.

The surface mixture is hauled to the work in trucks and is dumped on metal or wooden platforms or on the binder course, at a distance from the point of laying that will necessitate the entire load being shoveled into place. When shoveled into place, the mixture is thoroughly broken up with the rakes and is spread to the proper thickness for rolling.

FIG. 109.—Sheet-asphalt surface destroyed by concentrated traffic.

Mechanical spreaders are coming into use; and when they are employed, the mixture is spread to approximate thickness by hand or by a spreader attached to the rear of the truck. The mixture is then raked and spread to exact thickness by mechanical methods.

The rolling begins as soon as possible after the spreading, the first compression being secured by means of a light tandem roller and final compression by means of the 8- or 10-ton tandem roller. Here, again, the character and uniformity of the surface will depend to a considerable extent upon the skill of the roller operator. A uniformly smooth surface will result if the rolling is properly done; otherwise, unevenness is inevitable. Figure 107 shows the method of spreading the surface mixture, and Fig. 108 the rolling.

The surface is sometimes dressed lightly with cement or stone dust prior to the final rolling, although the necessity of this is a mooted point. For the average sheet-asphalt mixture, about 110 lb. of the mixture is required per square yard of surface, and, as with binder, the thickness is insured by computing the area that each load should cover and requiring the load to be spread in that area.

Smoothness.—To secure the required smoothness of surface, the accuracy of the spreading is checked by means of a straightedge applied to the surface parallel to the center line at various distances from the curb. As the rolling progresses, the checking is continued, and low areas are brought up by patches, while high areas that will not roll down are cut out and replaced. A good deal of the unevenness that develops in sheet-asphalt surfaces is attributable to uneven spreading and rolling and to lack of uniformity in the mixture itself. Areas with an excess of asphalt cement are quite likely to become noticeable depressions in a short time after the pavement is placed in service.

Some Features of Design.—If a street has sufficient longitudinal slope to insure that no water will stand along the curb, the straight concrete or stone-block curb may be used; but if the slope of the gutter is less than 0.5 per cent, it is better to design the street with the combined concrete curb and gutter or with a straight curb-and-block gutter.

It is undesirable to use sheet asphalt on a street where vehicles continually stand along the curb unless that portion of the street is paved with blocks of some kind. Wood-block, concrete, and grouted vitrified-brick surfaces are used for that part of the roadway.

The maximum grade for which the sheet-asphalt surface is permissible depends upon the kind of vehicles that use the surface. Instances of sheet-asphalt surfaces on grades up to 8 per cent have been noted, and no especial difficulty is reported.

Where street-car tracks occupy the middle portion of the street there is a decided tendency for traffic to concentrate on the strip of pavement between the track and the curb, especially if the track paving is rough or noisy. This can be overcome to some extent by paving the car-track area with sheet asphalt or wooden blocks. Where traffic concentrates on the sheet asphalt, creeping and unevenness quickly develop, and the pavement wears out very rapidly (see Fig. 109).

Characteristics of the Sheet-asphalt Surface.—The sheet-asphalt surface is compared with other pavements in another chapter. It is desired here to call attention to certain characteristics that are inherent. Sheet asphalt requires a certain amount of traffic to keep the asphalt "alive." It is noticeable that on light-traffic streets the part of the pavement next the curb becomes granular and sometimes cracks badly, whereas the middle portion is kept in good condition by the action of traffic. Sheet asphalt also deteriorates rapidly under concentrated truck traffic such as may be expected on a narrow street with a busy double-track car line.

FIG. 110.—Results of use of poor asphalt and improperly graded sand.

Stone-filled Sheet Asphalt.—Stone-filled sheet asphalt consists of a standard heavy traffic sheet-asphalt mixture to which is added about 30 per cent of hard stone chips of a size passing the ½- or ⅝-in. screen and retained on the 10-mesh sieve. This type of surface was originally known as *Topeka type asphaltic concrete* and was for years a competitor to the Bitulithic pavement. It was long customary to place the wearing surface of stone-filled sheet asphalt directly on the concrete base in the same manner as asphaltic concrete, but as traffic loads increased with the growth of truck transportation these surface courses began to exhibit considerable evidence of creeping. It will be apparent that the mixture is not well graded and in consequence does not have mechanical stability. When use in modern practice, a binder

course is provided, and the type is recognized for what it really is—a slight modification of standard sheet asphalt.

TABLE XXXV.—SIEVE ANALYSIS OF TYPICAL TOPEKA MIXTURE

	Sample no.			
	1	2	3	4
	%	%	%	%
Bitumen	8.00	6.0	10.0	7.0
Passing 200-mesh	15.6	10.4[1]	11.9	8.9[1]
Passing 100, retained on 200-mesh	13.3	14.5	11.1	9.7[1]
Passing 80, retained on 100-mesh	10.0	10.8	10.6	10.1
Passing 40, retained on 80-mesh	18.3	23.0	24.5	23.0
Passing 20, retained on 40-mesh	11.4	12.7	9.7	11.3
Passing 10, retained on 20-mesh	8.6	10.8	5.3	8.7
Passing 4, retained on 10-mesh	7.4	6.8[1]	8.0	9.7
Passing ½, retained on 4-mesh	7.4[1]	5.0[1]	8.9	11.6

[1] Indicates a deficiency in that size.

ASPHALT-BLOCK PAVEMENTS

The asphalt-block pavement surface is constructed of blocks of a mixture similar to that used for sheet asphalt, compressed by very heavy pressure. Since the blocks are made in a specially equipped factory, all of the details of manufacture are subject to close control, and the proportions of the various ingredients and the quality of the materials can be determined accurately. The product is, therefore, likely to be uniform. The blocks are formed under much greater pressure than can be secured in rolling a sheet-asphalt surface, and consequently they have greater density than has the average sheet surface.

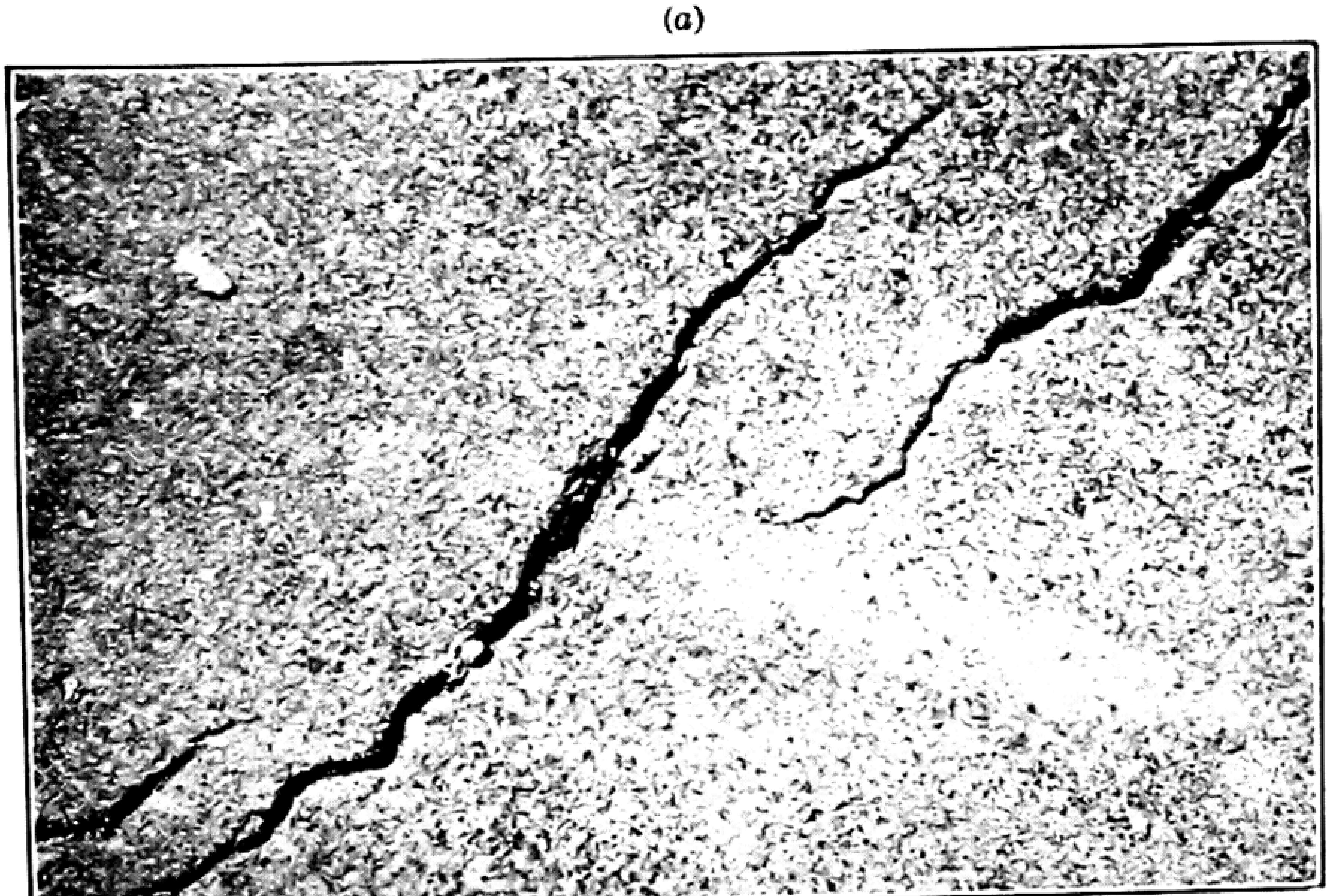
Composition of the Blocks.—Asphalt blocks are composed of graded crushed stone or sand, filler, and asphalt cement. A typical specification provides as follows:

Sieve Sizes	Per Cent
Retained on ¼-in. sieve, round opening	Not more than 3
Passing ¼-in. sieve and retained on 20-mesh	35–60
Passing 20-mesh sieve and retained on 100-mesh	15–30
Passing 100-mesh (including all fines)	20–35
Passing 200-mesh sieve	Not less than 15
Bitumen	6.5 to 9.0

Size of Blocks.—The blocks are made 2, 2½, or 3 in. thick and are 12 in. long and 5 in. wide. The 2-in.-thick block is

the one most widely used and is suitable for all ordinary locations. A variation of 1/4 in. from the specified size is permitted. The

(a)



(b)

Fig. 111.—Showing defective sheet-asphalt surfaces. In (a) the asphalt pavement is too soft and in (b) it is too hard. (Views about one-half natural size.)

blocks are formed under a pressure ranging from 240,000 to 360,000 lb. per block.

Subgrade.—The earth subgrade is prepared in the same manner as for the other types of pavement. This has been previously described.

Foundation.—The asphalt-block pavement is laid on a base of gravel, macadam, or concrete, each type being constructed in the manner already described.

Bedding Course.—The bedding course is generally a layer of sand 1 or 2 in. thick which is placed in the same manner as for the other types of block pavement.

Laying the Blocks.—The blocks are laid on the sand cushion in regular courses at right angles to the center line of the street with the joints broken by a lap of at least 4 in. Each block is driven against the course already laid by means of a heavy maul so as to give close transverse joints, and the end joints are tightened by means of a lever from the end of the course. The surface is covered with fine sand (passing a 20-mesh sieve). A plank or an iron plate is then placed over several courses, and the blocks are rammed to place by tamping through the medium of the plank or metal plate.

CHAPTER XVIII

THE ECONOMICS OF HIGHWAY TRANSPORTATION

The efficient management of a highway system requires among other things that the roadway surfaces provided for the use of the traffic be of such types and designs that highway transportation can be conducted at the lowest possible cost, when the cost of the highways to the public and the cost to the owners of vehicle operation are given due consideration.

The Cost of Transportation.—The cost of highway transportation is made up of two elements: the costs incurred on account of the highways and the costs incurred on account of the vehicles. These costs are interrelated in a very complex manner. It is possible that laudable but unwise attempts to decrease the cost of either will result in increasing the cost of the other and of their sum. By wise management of the highway system the sum of the two costs may be brought to the minimum consistent with reasonable convenience and safety. The direct cost of the highway is borne by the public as a whole through vehicle taxes, general taxes, and special assessments; and the indirect costs fall on the public and particularly those individuals who operate motor vehicles, whereas the cost of vehicle operation is borne by the individual owner of the vehicle. The determination of the program of highway improvement that will insure the lowest over-all cost involves consideration of both highway costs and vehicle operating costs. There is dearth of correlated data on vehicle operating costs on the several types of road surfaces; nevertheless the economic value of a road surface can be established only when the cost of vehicle operation thereon can be estimated with a fair degree of accuracy.

The cost of transportation involves more than the direct out-of-pocket money payments that represent the layman's conception of road costs or vehicle operating costs. The indirect costs that accrue are so completely concealed and illusive that only painstaking analysis will reveal them, yet they must be paid and are paid without that fact being recognized. The true cost of trans-

portation can be determined only when all the indirect costs are included, and they must be determined and provided for if the highway program is to be soundly financed.

The Value of a Highway.—Value is the worth or utility of anything, expressed in terms of a standard medium of exchange. The basis of value is the prospect of future returns in the form of money, service, or satisfaction. The returns receivable from a highway consist of service and satisfaction. The value of a highway is not measured by its cost new but rather by its annual cost during its useful life and by the cost of transportation thereon. The public permits its tax contribution to be invested in a highway because a return in the form of service and satisfaction is anticipated and is expected to be ample compensation for foregoing the use of the money in some cash-income producing investment.

It is reasonably safe to assume that if the cost of operating any vehicle on a road can be lowered by improving the wearing surface, a corresponding saving will be enjoyed by all other vehicles using that surface although not necessarily at the same rate. The value of a particular road type can then be estimated by comparing the saving effected in vehicle operating costs, determined for a few typical vehicles and then applied to the traffic as a whole, with the cost of securing that saving.

Unit of Measure of Highway Service.—The costs of automobile operation are usually expressed in terms of vehicle-miles; the costs of commercial vehicle operation, in terms of ton-miles; the road costs, in terms of square yards. If it becomes necessary to bring all these costs into a single unit, the only one of general application would be *cost per ton-mile per foot of width of highway.* But for some comparisons it is convenient to use cost per ton-mile per traffic lane or simply costs per ton-mile for a known highway. The important consideration is to employ a unit that can be compared with readily available records of cost, which are, for highways, in terms of either cost per square yard or cost per mile of specified design.

Useful Life of Road Surfaces.—The value of a particular highway or proposed highway can be estimated only when its probable service life can be predicted with reasonable accuracy. The useful life is dependent in part upon the traffic load and in part on climatic influences. The method employed in estimating probable service life is based on statistical data on the age at

retirement of road surfaces that have been replaced because of physical decrepitude or functional inefficiency.[1] The data on the service lives of pavement surfaces are not numerous or conclusive. Winfrey made an exhaustive study of the retirement statistics available in the records of highway departments;[2] Table XXXVI gives a summary of some of these, and typical road surface mortality curves based on the same data are shown

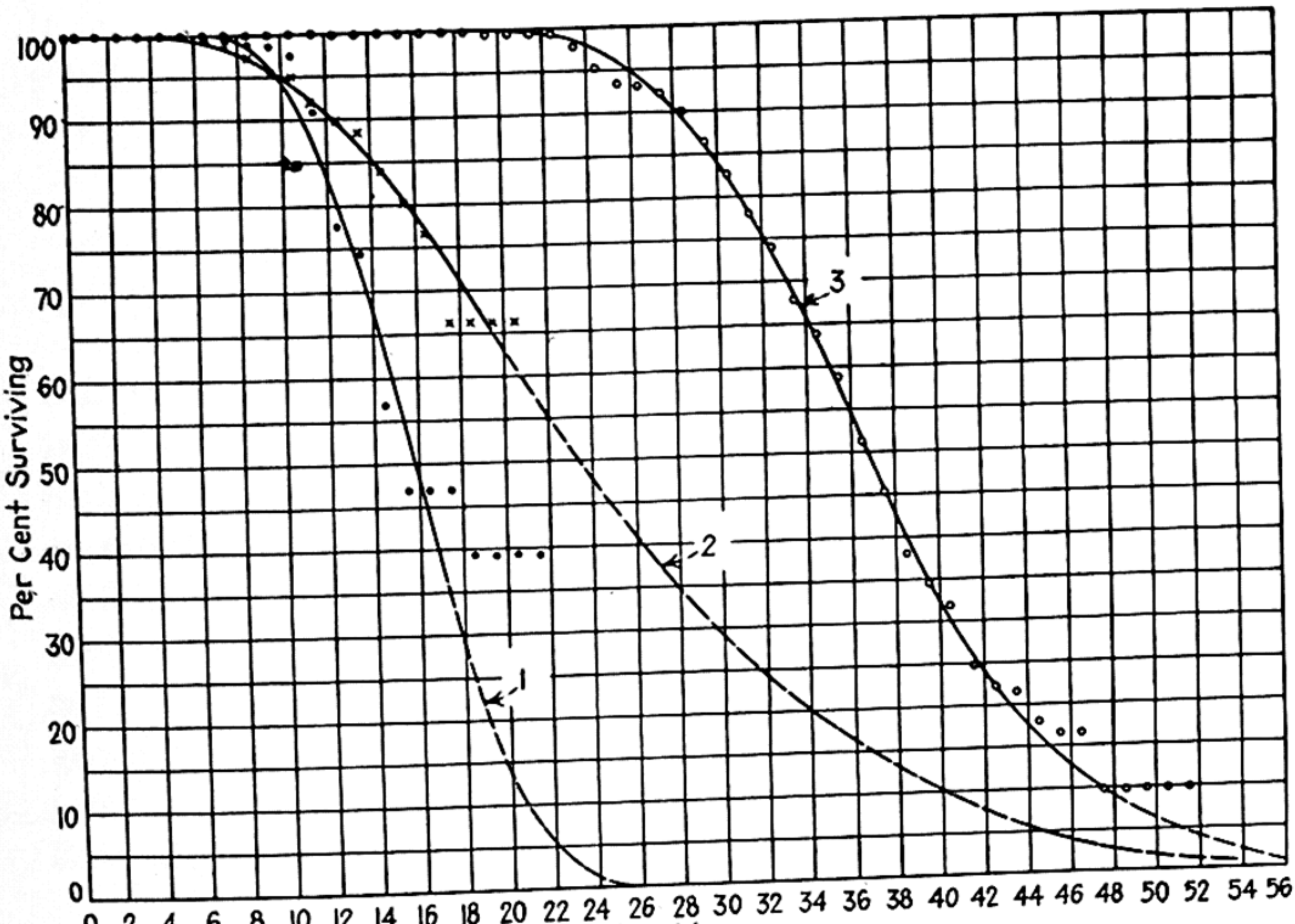


FIG. 112.—Showing typical road-surface mortality curves.

in Fig. 112. Although this information is not of general application, it is extremely useful as an indication of the limits of values for average life. When such data are required for use by a highway department, they should be taken from the department's own records, and the deductions made therefrom should be adapted to the problem under consideration according to the judgment of qualified persons familiar with the local conditions surrounding highway usage. In economic comparisons of the several types of road surfaces, the lowest reasonable values for

[1] MARSTON, ANSON, and T. R. AGG, "Engineering Valuation," McGraw-Hill Book Company, Inc., N.Y., pp. 48, 84, 1936.

[2] WINFREY, ROBLEY, "Preliminary Studies of the Actual Service Lives of Pavements," *Proc. 15th Annual Meeting*, Highway Research Board, 1935, pp. 53–54.

pavement life should be used because of the probable growth of traffic and possible rapid retirements on account of obsolescence.

Highway Transportation Costs.—The fundamental theory involved in the computations that will be used in any analysis of the character under discussion will be set forth briefly, but it should be noted that not all those competent to judge in matters of this nature are in substantial agreement with all the material presented herein.

The starting point in the development of the necessary theoretical basis for economic comparisons in highway work is the simple statement that:

$$\text{Highway transportation cost} = [\text{highway cost}] + [\text{vehicle cost}]. \tag{1}$$

The application of the principle thus set forth to a specific case requires that both groups of costs be ascertained and that they be reduced to a common unit so that they can be added to give a correct unit cost of transportation. Probably the most convenient unit for vehicle cost would be the ton-mile, which is used in railway practice; but the nature of highway traffic does not readily lend itself to the use of ton-mile costs. The use of the vehicle-mile has developed gradually as a measure of highway traffic; and although such a unit is open to criticism from several standpoints, it will be used herein because it is already well established and generally understood.

The highway costs that must be considered are all the costs of all kinds incident to the construction, maintenance, and administration of the highway. They may be expressed as follows:

$$\text{Annual highway costs} =$$
$$\begin{bmatrix}\text{annual return}\\ \text{on the value}\end{bmatrix} + \begin{bmatrix}\text{annual cost of}\\ \text{routine maintenance}\end{bmatrix} + \begin{bmatrix}\text{annual admin-}\\ \text{istration and}\\ \text{operating costs}\end{bmatrix}$$
$$+ \begin{bmatrix}\text{annual}\\ \text{depreciation}\end{bmatrix} + \begin{bmatrix}\text{annual cost of}\\ \text{periodic repairs}\end{bmatrix}. \tag{2}$$

The several items in Formula (2) require definition and an explanation of the fundamental basis for each.

1. *Return on Value.*—The sums invested in highway improvement are contributed by the public, each individual paying a share. Had the citizen retained his funds instead of paying them

to the commonwealth as a road tax, he could have invested them in cash-income-producing enterprises; and if all went well, he would have received a money return. The return that he foregoes in order to have the use of the highway is therefore a part of the indirect and, to him, unrecognized cost of highway service. The rate of return to employ in computing the cost of a highway is that which the individual might hope to secure on long-time investments. In 1939 that rate is probably about 4 per cent. The return base is the present value of the highway, which is equal to its cost new less the depreciation that may have accrued.

2. *Annual Depreciation.*—The depreciation of any element of a highway is the loss in value that accompanies age and use because of the approach of the date when the element must be retired, and each year's depreciation is a part of the cost of providing highway service that year. Depreciation must be estimated separately for right-of-way and earthwork, drainage structures, wearing surface, signs and other appurtenances, and any other element that constitutes a distinct service unit. Of all possible methods, the depreciation of any element of a highway is probably most satisfactorily computed by the straight-line, fixed-percentage, theoretical-depreciation method in which

$$\text{Annual depreciation} = \frac{(V_n - V_s)}{n}, \tag{3}$$

in which

V_n = the value new, in dollars.

V_s = the salvage value, in dollars.

n = the probable useful life of the element, in years.

And

$$\text{Accrued depreciation} = (V_n - V_s)\frac{x}{n}, \tag{4}$$

in which

x = the age at which the accrued depreciation is being computed.

The present value of the highway V_p is equal to the value new minus the accrued depreciation which, for straight-line depreciation, may be written:[1]

[1] Marston and Agg, *op. cit.*, p. 98.

$$V_p = V_n - \frac{(V_n - V_s)x}{n} + V_s. \tag{5}$$

and in a more general form may be written:[1]

$$V_p = (V_n - V_s)\left(\frac{\text{condition-per cent}}{100}\right) + V_s \tag{6}$$

Formula (6) holds true for any theory of computing accrued depreciation when the condition-per cents are calculated according to the particular theory of depreciation accountancy that it is desired to employ.

3. *Cost New.*—The cost new (V_n in Formulas (3), (4), (5), (6) and (7)) includes the direct or contract cost of construction, or the equivalent cost of force account work, plus the engineering and general overheads and "interest lost during construction."

Contract costs are well understood and require no discussion. The cost of force account construction includes the same items as contract cost except financing costs, bond costs (sometimes), liability insurance, and contractors' profit.

Engineering overhead includes the cost of surveys, plans, tests, and inspection, which are usually calculated as a percentage of the direct costs. General overhead includes the cost of advertising for proposals, expenses in connection with official meetings, general office expenses, legal expenses, and the like, which are also generally calculated as a percentage of the direct costs.

Interest lost during construction is an item included to compensate for the service denied during the period of waiting between the time when taxes are paid and the time when the highway is ready for use. This is frequently a difficult period to determine, particularly in those states in which the taxes are paid semiannually. In specific cases the period of waiting can be estimated with some degree of assurance; and if so, the return in compensation for the service foregone while waiting for the improvement should be added to the other items of cost. The rate of return is the same as is employed in paragraph 1, above.

4. *Cost of Operation.*—The cost of operation of a highway consists of those shares of the administrative and general costs incurred by the commonwealth or its subsidiary units in operating the highway system. These costs may for convenience be

[1] *Ibid.*, pp. 96, 102.

grouped into administrative costs and operating costs. The administrative cost consists of that portion of the highway department expenses that cannot be allocated to specific construction or maintenance operations. The cost of traffic control is a highway operating expense, and the portion of the cost of highway patrols and policing that is properly chargeable to traffic control should be included in the cost of highway operation.

Taxes in an amount equal to those assessed on privately owned property having a value equal to the present fair value of the highway V_p are a proper item of cost of operation of the highway. In economic studies it is permissible to calculate the tax charge on the basis of the average value of the road during its useful life. Of course no taxes are paid on the highway investment; but if the money had been invested in an industry, taxes would be paid. Therefore the inclusion of an item to compensate for taxes equivalent to those other investments pay is appropriate.

5. *Annual Maintenance.*—The annual maintenance cost includes overhead, labor, and supplies used in the routine maintenance of all elements of the highway. Typical maintenance operations are: filling the cracks in concrete or block pavements, trimming excess filler from expansion joints, painting traffic control lines, patching bituminous surfaces, dragging gravel or earth roads, cutting weeds, removing snow, and restoring drainage channels. The overhead costs of maintenance include supervision and a share of central office expenses. The cost[1] of the equipment should be included, and this is most readily accomplished by establishing a rental rate for each type used. The segregation of the maintenance costs of road surface from those of the right-of-way is sometimes quite difficult, because of the character of the records available, but average rates per mile per year for each can generally be established with reasonable accuracy.

6. *Periodical Maintenance.*—Some types of road surface are most effectively maintained by a general overhauling of some sort at intervals of several years. Resurfacing a penetration macadam at intervals of 3 or 4 years to supplement the regular patrol maintenance is an example. The appropriate share of this cost should be debited against the service each year, even though

[1] The correct determination of equipment costs involves knowledge of the economic life of the equipment and the annual cost of its use in accordance with the general principles of p. 432.

the actual expenditure is made only after a period of n' years. The distribution of this cost may be made on the straight average basis, the cost for each year being the total cost of periodical maintenance divided by the interval n'.

7. *Salvage Value.*—The salvage value V_s is the amount by which the utilization of the remains of the existing surface lessens the cost of reconstruction. The salvage value must often be estimated or approximated when the reconstruction is accomplished with a type of surface that is not identical with that which is being replaced. Salvage value may be positive, negative, or zero. After reconstruction, a new cycle of costs is established as follows:

$$V_n' = C_r + V_s,$$

where

C_r = the cost of reconstruction.

V_n' = the total investment after reconstruction.

Annual Road-cost Formula.—The relationships set up in (2) may be written as a formula. For purposes of illustration and to simplify the presentation, a formula[1] applicable to the road surface only is as follows:

$$C_s = \left[\frac{V_n - V_s}{2} + V_s\right]r + \frac{V_n - V_s}{n} + M_a + O + \frac{M_p}{n'} \quad (7)$$

C_s = the annual cost of any section of highway surfacing.

V_n = the cost new of the surfacing, including direct costs, overhead, and "interest lost."

r = the rate of return adopted for the analysis.

V_s = the residual or salvage value of the road surface when it is superseded.

[1] Formula (7) is based on straight-line depreciation and the elimination of the consideration of a sinking fund for accumulating the sum required for periodical maintenance. The theoretically exact form would make use of the fictitious sinking-fund conception for determining the annual depreciation and the annual cost of periodical maintenance, and the annual return on investment would be calculated on the cost new. The formula would be as follows:

$$C_s = V_n r + \frac{(V_n - V_s)i}{(1+i)^n - 1} + \frac{M_p i}{(1+i)^{n'} - 1} + M_a + O,$$

in which the several terms have the same definition as when used in Formula (7), and i is the rate of interest assumed for the sinking funds. i might be equal to r, or it might be less.

M_p = the cost of periodical maintenance.
M_a = the annual cost for routine maintenance.
O = the annual general administrative and operating costs chargeable to the highway in question.
n = the economic life of the surface, in years.
n' = the interval between periodical maintenance operations, in years.

Economic Life.—Statistics of actual retirements were employed in preparing the mortality curves shown in Fig. 112, and consequently the average lives deduced from these mortality curves

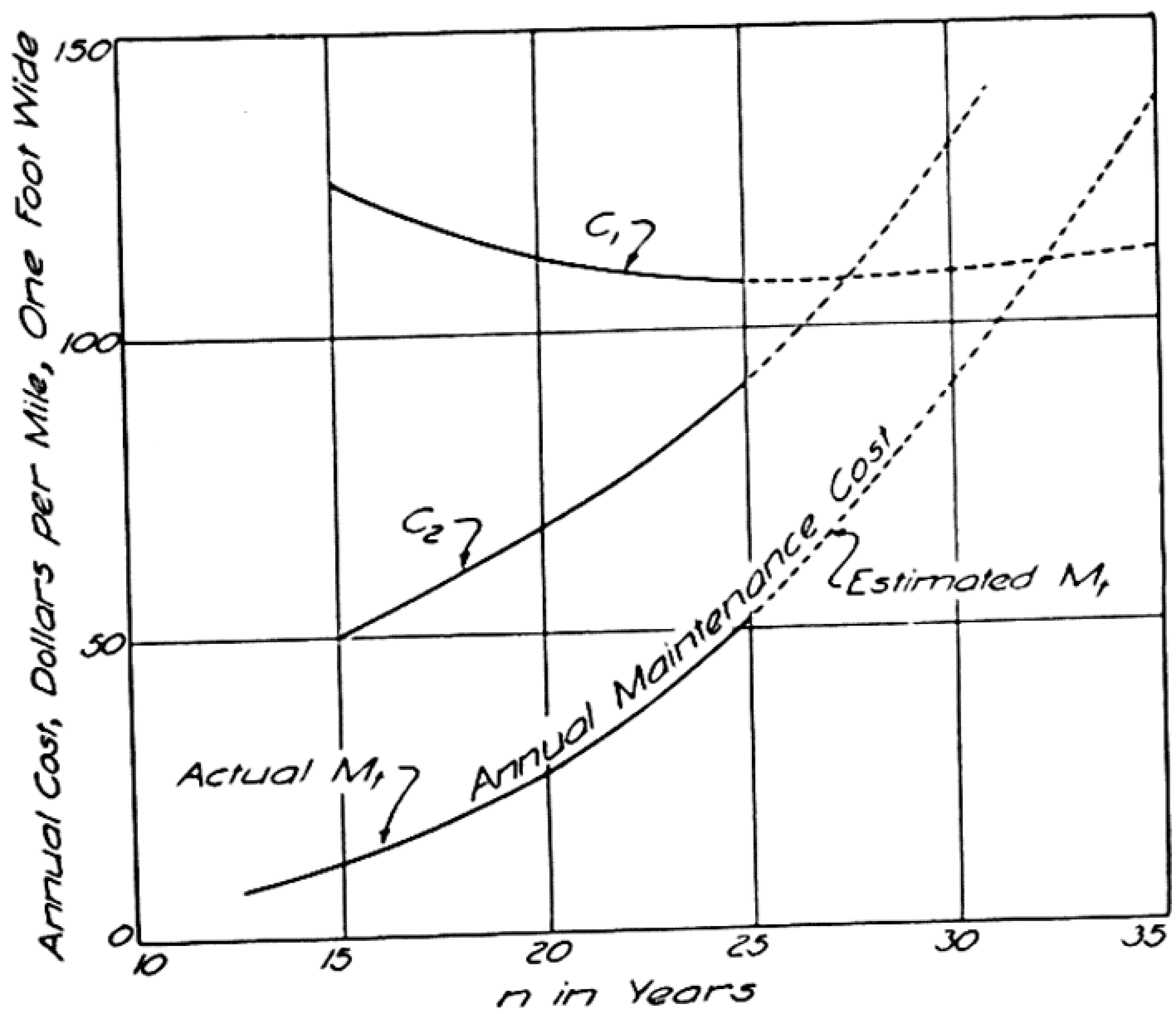


FIG. 113.—Illustrating determination of economic life.

are statistically correct, but there is no evidence to show that these road surfaces were actually retired at an age identical with their economic life. The economic life of a roadway surface is the period during which it must be kept in service for the annual cost to reach the minimum.[1] For the computation of economic life, Formula (7) may be simplified somewhat. Administration and operation expenses do not vary with the age of the road surface and may therefore be disregarded. The maintenance cost per year may be set up by adding together the cost of annual routine maintenance and the annuities required for the periodical maintenance. The simplified form of Formula (7) would then become

[1] For a more complete discussion of this subject see *ibid.*, p. 85.

$$C_1 = \left[\frac{V_n - V_s}{2} + V_s\right]r + \frac{(V_n - V_s)}{n} + M_a + M_p.$$

When the road surface has reached its economic life, it has salvage value only, and the annual cost of any service beyond that date is

$$C_2 = rV_s + M_p + M_a.$$

If the records of cost for a road surface permit estimating $M_a + M_p$ for some years in the future, C_1 and C_2 can be calculated for a series of values for n and plotted as shown in Fig. 113. The economic life is at the intersection of the curves for C_1 and C_2.

Annual Costs of Right-of-way and Appurtenances.—The annual cost of each of the highway elements is determined in like manner. In the usual case these are: right-of-way, earthwork and other improvements to right-of-way, drainage structures, and signs and other appurtenances.

1. *Right-of-way Costs.*—The rights-of-way for established highways are not usually owned in fee simple by the public; on the contrary, the public has an easement for the use of the land for highway purposes. The land required for the right-of-way needed for relocation, widening, and grade separation is usually purchased and therefore actually owned by the public. In any case the equivalent of a fair return on the value of the land employed for right-of-way is an element of the cost of furnishing highway service. If there were no highway, either the land or its money equivalent would have been in the hands of individuals for employment in enterprises affording a money return. The land might have been farmed, for example. The basis for the computation of the return to be charged against the cost of highway service is the present value of the land in right-of-way as evidenced by the present market value of similar lands adjacent.

There is also a tax loss to the commonwealth because the land has been taken out of usage that produces a money return, but such tax loss applies only to lands for which the public owns the fee. Those upon which the public holds an easement are included for taxation in the tracts from which they were originally separated.

2. *Cost of Right-of-way Improvements.*—The right-of-way is improved by grading; draining, including ditches, tile lines, and culverts; and landscaping. These improvements are in general of such long life that they may be considered to be kept in standard serviceable condition by maintenance. The depreciation factor is usually insignificant and for most purposes may be neglected.

The principal item of cost is usually that of earthwork. The cost of earthwork includes direct or contract costs, general and engineering overhead, and "interest lost during construction." The annual cost of earthwork includes return on present value, maintenance, a share of the operation costs, and depreciation if considered significant. The element of taxes foregone must also be evaluated.

Similarly the annual cost of drainage facilities, except bridges and landscaping, must be included.

3. *Bridges.*—The cost of culverts may conveniently be included in the cost of improvements in the right-of-way, but major stream crossings require structures of such great cost that to allocate all of it to a short length of highway would manifestly be unfair. Generally the annual cost of any one of these major bridges can be allocated to the roads that lead to it and lie within say 25 or 50 miles according to circumstances. The great bridges across such streams as the Hudson, Mississippi, Ohio, Tennessee, Colorado, Missouri, and Columbia rivers should be allocated to connecting roads for much greater distances, perhaps several hundred miles in some cases.

The annual cost of major stream crossings is determined as for the other elements and includes return, depreciation, maintenance, and operation including taxes.

4. *Signs and Similar Appurtenances.*—The annual cost of such items as guide and warning signs, guard fence or walls, parking or camping areas, and similar conveniences may be considered as made up of return, maintenance, and operation, including taxes. The item of depreciation is so enmeshed with maintenance that the two cannot be separated.

Example of Calculation of Costs.—The several terms employed in Formula (7) have been discussed on pages 434–438, and it remains only to interpret them for the calculation of the annual cost of the several elements of this highway.

1. *Pavement costs.*—The data for the two-lane concrete pavement having a length of 26 miles are as follows:

Contract cost	$771,000	
Overhead at 6.13%	47,262	
"Interest lost" during construction at 4% for 6 months	15,420	
Total cost new of pavement = V_n		$833,682
Annual maintenance cost	9,850	
Maintenance overhead at 0.4%	40	
Total maintenance = M_a		$ 9,890
Salvage value, V_s, estimated		368,000
Periodical maintenance		0
Operation including traffic control, O		3,150
Age of pavement at 9 years, = x		
r = 4%. Probable life of pavement, 25 years, = n		

The annual cost of the pavement is as follows:

$$\text{Annual depreciation} = \frac{V_n - V_s}{n} = \frac{465{,}682}{25} = \$18{,}627.$$

$$\text{Average value } \frac{V_n - V_s}{2} + V_s = \frac{465{,}682}{2} + 368{,}000 = \$600{,}840.$$

Tabulation of items comprising C_s

Return on average value, at 4%	$ 24,033	
Annual depreciation	18,627	
Annual maintenance	9,890	
Annual operation costs, *pro rata* share state costs	3,150	
Taxes foregone, based on the average value	10,200	
Total annual cost		$ 65,900
Annual pavement cost per mile	$ 2,535	

2. *Right-of-way costs:*

The right-of-way is an easement and has an area of 217 acres. The adjacent land has a present market value of $90 per acre. No tax allowance required because the land is used by virtue of an easement.

Right-of-way value, (217)(90)	$ 19,530	
Annual return on value at 4%	781	
Annual operation costs, *pro rata* share state costs	98	
Annual cost of right-of-way		$ 879

3. *Cost of earthwork and minor drainage structure:*

Contract cost of earthwork	$178,650
Overhead, at 6.13%	10,950
Interest during construction, at 4% for 6 months	3,570
	$193,170
Contract cost of drainage structures	$152,185
Overhead, at 6.13%	9,328
Interest during construction, at 4% for 6 months	3,042
	$164,555

Average value of earthwork and minor drainage structures	$275,450	
Annual depreciation of earthwork, none		
Annual depreciation of minor drainage structures on the basis of n = 100 years and $V_s = 0$	$ 1,645	
Annual return on average value of earthwork and minor drainage structures, at 4%	11,018	
Annual maintenance of earthwork and minor drainage structures including maintenance of right-of-way	2,930	
Annual operation costs, *pro rata* share state costs	780	
Taxes foregone, based on the average value	5,900	
Total annual cost for earthwork and minor drainage structures		$ 22,270

4. *Costs of signs and other appurtenances:*

Original cost of construction including overhead, $4,780. Service life continuous under adequate maintenance.

Annual maintenance cost	$ 269	
Return on original cost, at 4%	$ 191	
Annual operation costs, *pro rata* share state costs	24	
Taxes foregone, based on original cost	9	
Total annual cost of signs, etc		$ 493

5. *Summary of annual costs:*

Pavement		$ 65,900
Right-of-way		879
Earthwork and drainage		22,270
Signs, etc		493
Total annual cost		$ 89,542
Annual cost per mile	$ 3,444	

VEHICLE COSTS

The true cost of operating a motor vehicle includes the well-understood out-of-pocket money payments and certain concealed costs which accrue in much the same way as the concealed cost of the highway and are revealed only when the cost of motor vehicle operation is subjected to an analysis similar to that employed for estimating highway costs.

Elements of Vehicle Costs.—The cost of owning and operating a motor vehicle can be estimated by utilizing a cost formula identical in principle with Formula (7). It is

$$C_v = \left[\frac{V_n - V_s}{2} + V_s\right]r + \frac{V_n - V_s}{n} + M_a + O, \tag{8}$$

in which

C_v = the annual cost of vehicle usage.

V_n = the cost new.

V_s = the salvage value at the end of the useful life (or the "trade-in" value for any one owner, who is computing the cost of service during his use of the vehicle).

n = the useful life of the vehicle.

M_a = the annual cost of maintenance.

O = the annual operating costs: fuel, oil, taxes, garage, insurance, etc.

r = the rate of return adopted for the computations.

The several elements of cost are readily recognizable but require correct handling in economic analyses.

1. *Return on Investment.*—The corporation that operates a fleet of busses would obviously expect to earn a return on the investment in vehicles. The business that furnishes automobiles to its salesmen would likewise expect a return on the investment in automobiles. The individual owner of an automobile receives service and satisfaction from the use of a car in lieu of a money return. He might have invested the price of the car in a bond or share in some business and have received a money return therefrom. He accepts the use of the automobile instead, but a part of his cost of ownership is the return that he foregoes on his investment. The rate of return to employ in estimating the return foregone is the rate that he could hope to secure on appropriate investments, probably about 4 per cent in 1939. The return should be computed on the value of the vehicle at the beginning of the year, but for economic analyses it is sufficiently accurate to compute the return each year on the average value of the vehicle during its useful life.

2. *Depreciation.*—The depreciation of the vehicle is a function of age and use but for the average automobile is largely a question of age, since the process of selling automobiles involves the "trade-in" practice, and the trade-in allowance is primarily a matter of age. However, in studies of transportation costs the significant factor is the total depreciation and the average per year, not the depreciation under any one ownership. Here, again, the straight-line depreciation method may be employed.

3. *Maintenance.*—This factor requires no especial consideration, since it is well understood. The average yearly maintenance expenditures should be determined and used in the

formula. The cost of tires and batteries is usually considered a maintenance expense, although some prefer to carry these items as a part of the operating costs.

4. *Operating Costs.*—These include fuel, grease, oil, taxes, garage, insurance, and cleaning. They constitute the familiar out-of-pocket costs which together with maintenance costs are usually thought of as the total cost of vehicle usage.

Vehicle Operating Costs.—The automobile is used on all sorts of roads, and except in unusual cases the owner has no means of judging the influence of the road surface on vehicle operating costs. The actual cost of operation on any road surface will reflect the physical characteristics of that surface, such as firmness, smoothness, and grittiness. The relation between the nature of the road surface and the cost of vehicle operation thereon has been studied and reported upon repeatedly[1] and is under investigation in several places at the present time. The data at hand on automobile operating costs are reasonably conclusive, but the costs of commercial vehicle operation are not so well established.

It seems clear from the investigations reported that certain direct costs are affected by road condition. They are

1. Fuel and oil costs.
2. Tire costs.
3. Maintenance costs.

It is also clearly established that the average speed is markedly affected by the type and condition of the road surface irrespective of other road factors affecting speed. The time lost because of inferior road conditions runs into very large figures for traffic as a whole, but there is no accepted basis for evaluating time lost. In many cases the time lost has no value, whereas in the case of business traffic it has a very substantial value. There are as yet no data showing the percentage of automobile traffic that could be classed as business traffic of a character justifying a charge against the highway for time lost.

The following computations are based on actual recorded automobile operating costs for a typical vehicle which cost $800 when new and for which the salvage value at the end of its economic

[1] The reports of the annual meetings of the Highway Research Board, Washington, D.C., have carried discussions of this subject each year for the past 10 years or more.

life of 8 years is so small that it can be neglected. Return is calculated at the rate of 4 per cent. The annual mileage is 8,000, and average rates are used for insurance and garage, since these costs vary greatly among operators.

1. *Return.*—The average value is $400 during the life of the vehicle and the return is therefore $16 per year, or 0.20 cts. per mile.

2. *Depreciation.*—The cost of $800 must be recovered in 8 years; therefore the depreciation is $100 per year, or 1.25 cts. per mile.

3. *Taxes and Fees.*—These amount to $13.57 per year for this type of vehicle in Iowa, or 0.17 cts. per mile.

4. *Insurance.*—The cost of theft, fire, and public liability insurance in Iowa for this vehicle would be $27.25 per year, or 0.34 cts. per mile.

5. *Garage.*—Garage is included at the average rate $2.50 per month, or $30 per year, which gives a cost of 0.38 cts. per mile.

The direct operating costs as well as the foregoing indirect costs are shown in Table XXXVII. This table also shows the differentials in operating costs on the three classes of road surface included in the table.

The annual and per mile cost of operating a truck or bus is determined in precisely the same manner as are similar costs for automobiles. The question of the propriety of including a charge for return on investment and for depreciation does not arise in the consideration of the cost of operation of commercial vehicles, as it is self-evident that those factors should be included. The volume of authoritative data on the actual cost of commercial vehicle operation is so inadequate that no comprehensive discussion of the subject is possible. For the illustrative problem that follows, the cost of commercial vehicle operation will arbitrarily be taken at 5 cts. per ton-mile for travel on paved roads.

Highway Transportation Costs.—The cost of highway transportation is made up of the annual cost of the highway plus the annual cost of operating the vehicles on that highway minus the contributions made by the vehicle to road funds through the license fee, gasoline tax, and any other wheel tax that may be imposed. Not all the tax contribution from the vehicle finds its way into road funds, and in many states it probably would be exceedingly difficult to determine exactly how much the vehicle

contributes to road funds. Nevertheless, the best estimate possible should be made in a specific instance, and proper deductions made in computing highway transportation costs.

In the illustrative problem an analysis was made of the contribution by the vehicle to road funds, and it was determined that approximately 3 per cent of the vehicle operating costs were

TABLE XXXVI.—SUMMARY OF CALCULATED AVERAGE LIVES[1]

Surface type	Group no., road or street system	Estimated average life, years
Brick on concrete base	1	22.0
	2	24.5
	3	36.0
	4	49.0
Bitulithic on concrete base	1	28.0
Sheet asphalt on concrete base	1	27.5
	2	46.0
	3	36.0
Sheet asphalt on concrete base (without railroad track)	4	36.5
	5	14.0
	6	32.5
	7	39.5
	8	41.0
	9	27.0
	10	34.0
Sheet asphalt on concrete base (with railroad track)	11	14.0
	12	18.0
	13	23.0
	14	34.0
	15	35.0
	16	36.0
Block stone on concrete base	1	38.0
Portland-cement concrete	1	13.7
	2	21.5
	3	29.0
	4	14.0
	5	16.0
	6	21.0
	7	12.0
	8	18.0
	9	16.0
	10	25.0
Bituminous concrete (coarse aggregate) on gravel or macadam base	1	18.0
	2	17.0
	3	17.0
Bituminous concrete (graded aggregate) on gravel or macadam base	1	15.0
Bituminous macadam penetration on gravel or macadam base	1	16.0
	2	14.5
	3	17.0
Bituminous macadam oil	1	15.5
Bituminous macadam-tar penetration on gravel or macadam base	1	17.0
	2	17.0
	3	18.0
	4	17.5
	5	18.0
Bituminous macadam asphalt penetration on gravel or macadam base	1	16.0
	2	14.0
	3	15.0
	4	17.5
	5	17.0
Bituminous gravel or macadam base	1	16.0
	2	14.5
	3	17.0
Water-bound macadam, surface treated	1	22.0
Surface-treated gravel	1	14.0

[1] WINFREY, *op. cit.*, p. 54.

TABLE XXXVII.—TYPICAL AUTOMOBILE OPERATION COSTS ON VARIOUS SURFACE TYPES IN IOWA

Cost item	Cost, cents per mile		
	Pavement	Gravel	Earth
Gasoline	1.22	1.40	1.35
Oil	0.09	0.15	0.21
Maintenance	0.10	0.61	1.24
Tires	0.26	0.38	0.34
Total of items varying with road condition	1.67	2.54	3.14
Garage	0.38	0.38	0.38
License fee and taxes	0.17	0.17	0.17
Depreciation	1.25	1.25	1.25
Return at 4% on an investment of $400	0.20	0.20	0.20
Insurance at Iowa rates	0.34	0.34	0.34
Total costs	4.01	4.88	5.48
Differential costs, type to type	....	0.87	0.60
Differential costs, low to high	....	....	1.47

in the form of taxes that found their way into the funds from which the highway in question was constructed.

The annual traffic on the highway was determined by a traffic census which showed that the road was used by 900,000 automobiles per year and by 300,000 commercial vehicles. The cost of transportation on this highway may therefore be summarized as follows:

	Number	Tons	Operation Cost per Mile
Automobiles per year	900,000	1,620,000	$ 36,000
Commercial vehicles per year	300,000	1,275,000	63,750
Total vehicle operating cost per mile of road per year			$ 99,750
Deduct contribution to road funds by vehicle operator			2,992
Net vehicle operating cost per mile of road per year			96,758
Annual highway cost per mile			3,444
Total cost of transportation per mile per year			$100,202

If it were assumed that the road costs should be divided between the two classes of vehicles according to the ton-miles of use, the road costs for the automobile would be $1,927, and the average cost of automobile transportation would be

36,000/1,620,000, or 2.22 cts. per ton-mile. On the same basis the average cost of commercial vehicle operation would be 4.97 cts. per ton-mile.

THE TRAFFIC CENSUS

The wise management of a highway system requires among other things that the responsible officials have accurate information as to the traffic on various highways under their control. The designs for new construction should be predicated upon known traffic; the plans for future financing depend upon estimates of future traffic; and the maintenance system must be developed to meet known traffic conditions. The traffic census provides the data upon which planning must be based.

The Traffic Census.—The traffic census is the process by means of which the volume and composition of the traffic stream are determined.[1] The assembly of the facts is a simple matter once the organization for doing the work has been developed, but at best the process requires time and a very carefully planned organization.

If it were necessary to count all the vehicles on every highway for long periods of time in order to establish the facts about traffic, the cost would be enormous, and the mass of data so great that its analysis and tabulation would become an almost hopeless task. Fortunately it has been discovered that the traffic pattern on a highway changes but slowly from year to year, and in fact the traffic pattern once established for a region may be considered applicable to all the roads in the region.

The Behavior of Traffic.—The investigations of the behavior of traffic show that for most highways the normal weekday traffic—Monday to Friday inclusive—when once determined may be taken as the traffic pattern for the entire region. If on one road the traffic from 9:00 to 11:00 A.M. on Tuesday is 20 per cent of the 24-hr. traffic passing that station, it can be assumed that the 9:00 to 11:00 A.M. traffic on any other road in the area will be 20 per cent of the 24-hr. traffic on that highway. Moreover it has been discovered that in regions in which no special conditions are encountered, the percentage of the traffic passing a counting station between 9:00 and 11:00 A.M. on a normal weekday will not fluctuate appreciably from season to season.

[1] McClintock, Miller, "Short Count Traffic Surveys," *Bull.* 3, *Highway Planning and Design Ser.*, Portland Cement Association, Chicago, Ill.

Exceptions to this rule are the highways in summer resort or vacation areas where July and August will show high rates of travel, and regions where snow and sleet are encountered during the winter months, and consequently the winter traffic pattern may differ greatly from the summer traffic pattern.

Establishing the Traffic Pattern.—The traffic pattern[1] is established by taking 24-hr. traffic counts at several control stations in an area, usually a county—at locations selected so as to avoid any short-distance local traffic that might be due to

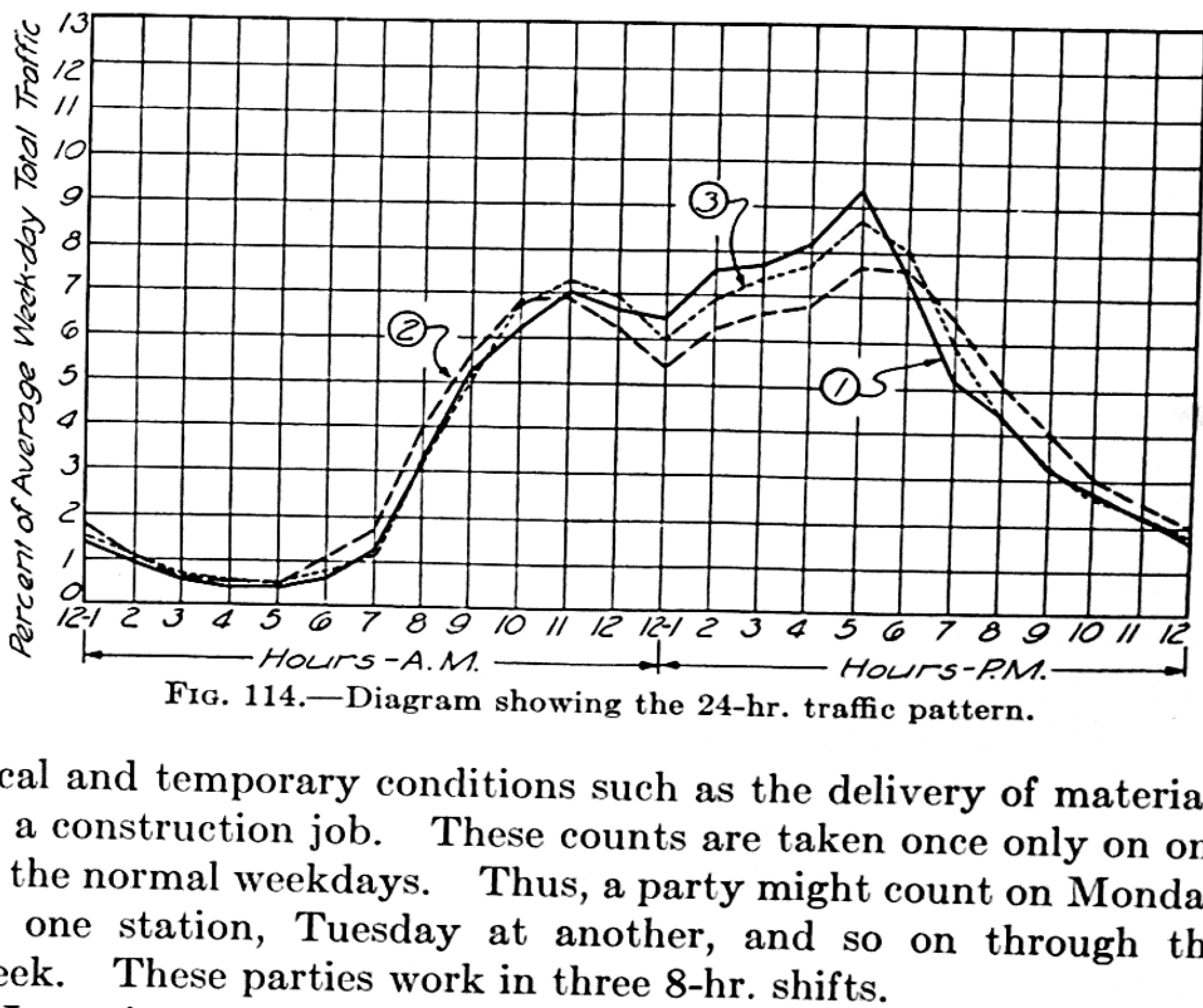


Fig. 114.—Diagram showing the 24-hr. traffic pattern.

local and temporary conditions such as the delivery of materials to a construction job. These counts are taken once only on one of the normal weekdays. Thus, a party might count on Monday at one station, Tuesday at another, and so on through the week. These parties work in three 8-hr. shifts.

In regions where there are no conditions that cause seasonal variations in traffic, which is true in large part of the United States, these counts may be made at any time of the year; but where seasonal variations are suspected, winter season counts[2] are taken also. Likewise where summer traffic is exceptionally

[1] Cherniack, Nathan, "Methods of Estimating Vehicular Traffic Volume," *Proc.* Highway Research Board, p. 253, 1936.

Shelton, W. Arthur, "Methods of Estimating Highway Volume," *loc. cit.*, p. 413.

[2] Morris, Mark, "Master Traffic Count on U.S. Highway 65," *Proc. 15th Annual Meeting*, Highway Research Board, 1935, p. 300.

heavy, as on U.S. 65 in Minnesota for example, summer and fall counts should be taken as well as winter counts.

The results of these control counts are tabulated to show the percentage of the automobile traffic passing the counting station during each hour of the 24. Similar counts are made for commercial vehicle traffic.

The composition of the traffic is established by recording the number of automobiles passing the station and the number of commercial vehicles of each desired weight classification. This latter must be secured by stopping the trucks and busses and securing the required information from the driver.

Routine Traffic Surveys.—The routine traffic survey is a 2-hr. count taken once only on a normal weekday between 9:00 and 12:00 A.M. and 1:00 and 4:00 P.M. It is apparent that such counts can be taken at a large number of stations at relatively small cost. By using standard forms and the careful training of the personnel, inexperienced help can be used for the work.

The gross traffic at a station is estimated by assuming that the volume of traffic of each class at a counting station during a counting period bears the same relation to the total traffic at that station as traffic during the same period at a control station bears to the total traffic at the control station.

Estimating Total Traffic.—With the data from the control and base station counts at hand, the steps to be taken in estimating the total yearly traffic are as follows.[1]

1. Determine a *standard multiplier* from the control station 24-hr. counts by adding together the percentages of the traffic passing the station for the 6 hr. 9:00 to 12:00 and 1:00 to 4:00 and dividing by 3. This gives the average percentage of the 24-hr. traffic passing the control station in 2 hr. of a normal weekday. Divide 100 by this 2-hr. percentage, and the result will be the standard multiplier.

2. Multiply the number of vehicles counted at the base station in 2 hr. by the standard multiplier, and the result will be the average weekday traffic. If it has been shown that the week-end traffic has characteristics differing from the normal weekday traffic, the weekday average is increased or decreased according to the facts observed to establish a factor for week-end traffic.

3. Calculate the total annual traffic by

a. Multiplying the average daily traffic of each class as computed in paragraph 2 by 365 if no correction is to be made for week-end traffic. This is sufficiently accurate for most areas.

[1] McCLINTOCK, MILLER, *loc. cit.*, p. 26.

Form No. HPS-1 [illegible]

Iowa State Highway Commission
U. S. Bureau of Public Roads

HIGHWAY PLANNING SURVEY

LOCAL STATION TRAFFIC DENSITY REPORT

County (Name)__________(No)____Sta. No.____Location of Station____Cor., Sec.____Twp.____Range____Weather____Date____193__Day____

Route No.________To________Direction from Station________

		LOCAL VEHICLES											FOREIGN VEHICLES										
			TRUCKS								BUSSES			TRUCKS									
			Mfr's Rated Capacity in Tons											Mfr's Rated Capacity in Tons									
Hours	Horse Drawn Vehicles	Passenger Cars	Under 1½	1½	Over 1½ and Under 3	3 and Under 5	5	Over 5	Tractor Semi-Trailers	Trucks With Trailers	School	Other	Passenger Cars	Under 1½	1½	Over 1½ and Under 3	3 and Under 5	5	Over 5	Tractor Semi-Trailers	Trucks With Trailers	Busses	

Form No. HPS-2—Revised 7-3-36
[illegible]

Iowa State Highway Commission
U. S. Bureau of Public Roads

HIGHWAY PLANNING SURVEY

PRIMARY BLANKET AND KEY STATION TRAFFIC DENSITY REPORT

County Name________No.____Sta. No.____Location________Weather________Date____193__Day____

ROUTE NO.________TO________DIRECTION FROM STATION________

	LOCAL VEHICLES								FOREIGN VEHICLES							
		Trucks					Busses			Trucks						
Hours	PASSENGER CARS	Light	Medium	Heavy	Tractor Semi-Trailers	Trucks with Trailers	School	Other	PASSENGER CARS	Light	Medium	Heavy	Tractor Semi-Trailers	Trucks with Trailers	Busses	Total Motor Vehicles

FIG. 115.—Typical forms for recording data at traffic counting stations.

Form No. HPS-4 ([illegible])

IOWA STATE HIGHWAY COMMISSION—U. S. BUREAU OF PUBLIC ROADS

HIGHWAY PLANNING SURVEY

TRUCK AND BUS LOADOMETER REPORT

Station Number________________

Sheet Number__________of__________

Location of Station________________

Party Number________Weather________

(Signed)________________, Recorder

Date__________, 193___Day________

Hour Period of Count: From________to________

1	2	3	4	5	6	7			8			9	10	11	12	13	14			15	16	17	18	19
						Origin			Destination								Wheel Loads							
State of Registration	Registration Class	License Number	Type of Vehicle	Type of Body	Rated Capacity or Seating Capacity (Tons or No. of Persons)	City or Town	County	State	City or Town	County	State	Trip Mileage	Type of Origin	Type of Destination	Commodities Carried or Number of Passengers	Operating Classification	(1)	(2)	(3)	Loaded Weight	Empty Weight	Carried Load	Total Weight of Combination	Empty Weight Plus Licensed Capacity

Form No. HPS-7—[illegible]

IOWA STATE HIGHWAY COMMISSION

U. S. BUREAU OF PUBIC ROADS

HIGHWAY PLANNING SURVEY

TRUCK AND BUS PIT SCALE REPORT

Station Number________________

Sheet Number__________of__________

Location of Station________________

Party Number________Weather________

(Signed)________________Recorder

Date__________193___Day________

Hour Period of Count: From________To________

1	2	3	4	5	6	7	8	9	10	11	12		13	14	15	16	17	18	19
											Length								
State of Registration	Registration Class	License Number	Make and Year	Type of Vehicle	Type of Body	Manufacturer's rated Capacity or Seating Capacity (Tons or No. of Persons)	Commodities Carried or Number of Passengers	Tire Equipment	Tire Size (Inches)	Overall Width (Feet)	Overall (Feet)	Wheelbase (Feet)	Maximum Height (Feet)	Rear Axle Weight	Loaded Weight	Empty Weight	Carried Load	Total Weight of Combination	Empty Weight Plus Licensed Capacity

FIG. 116.—Forms for recording data at traffic weighing stations.

b. Or multiplying the average daily traffic of each class for a normal weekday by 313 and the average week-end traffic of each class by 52 and adding the two sums. This is required only where the week-end traffic differs markedly from the normal weekday traffic.

c. Or multiplying the average daily traffic of each class as computed in paragraph 2 for each season by the number of days in the season and adding the sums to secure the total annual traffic. This is necessary where it has been determined that the traffic for any season is markedly different from that of another.

The traffic patterns shown in Fig. 114 determined as described above are quite illuminating. They were determined as follows:

Curve 2 is the average daily traffic as determined by making 24-hr. counts for one year.

Curve 1 is the average daily traffic based on 24-hr. counts at 151 stations.

Curve 3 is the average daily traffic in the winter months based on counts for 16 weeks in the winter.

The forms used for recording the traffic in an Iowa traffic survey are shown in Fig. 115. The "primary blanket and key stations" are control stations at which 24-hr. counts were made. The "local stations" were base stations.

Traffic Weighing Stations.—The volume of traffic is determined from traffic census records taken at counting stations, but the weight of the traffic loads can be ascertained only by actually weighing a sample of the traffic. This may be done by means of loadometers, which can be set up quickly and, after a fair sample has been weighed, moved to another weighing station. Some stations are used both for weighing traffic as a traffic survey operation and also for detecting overloaded trucks, and for that purpose a platform scale is installed in a semipermanent manner. The weighing station is used as a means of collecting a mass of data on the equipment and dimensions of the vehicle, the nature of which is best illustrated by the forms for weighing station reports shown in Fig. 116.

HIGHWAY DEPARTMENT ACCOUNTING[1]

In keeping with the general practices followed by offices of governments, state and county highway departments have kept records of receipts and disbursements but have given little or no consideration to cost accounting and investment records, which are an accepted element of the accounts kept by industrial and

[1] This section in slightly expanded form contributed by Robley Winfrey.

commercial organizations. The time has arrived when it is essential to expand highway accounting to include cost accounting to include not only transactions by the highway department itself but also those accruing in operating and owning the highway system.

Functions of the Accounting System.—Accounting will aid the highway department in keeping itself solvent, in determining the amount of revenue that it must receive in order to perpetuate the highway system, in effecting economy in its various operations, and in determining the over-all expense to be allocated to the various classes of highway users and other beneficiaries of highway improvement.

The accounting section of a state highway department should probably rank with the sections dealing with construction, maintenance, equipment, and research and should have the authority and prestige required to control finances and perform proper fiscal and cost accounting. This accounting section should have supervision of expenditures and the operation of the budget set up by the commission or other designated authority, and no obligation should be incurred without first ascertaining that allocated funds are available for payment. The accounting section, therefore, must be coordinated with the other sections of the highway department. The functions of the accounting section are primarily three in number: (1) fiscal accounting, (2) cost accounting, and (3) statistical compilation and analyses thereof.

Fiscal Accounting.—Fiscal accounting is concerned primarily with keeping records of the various incomes by sources and the expenditures by appropriations and funds according to the purposes designated by the legislature or highway commission and with budgetary accounting by which the administration prevents any operating unit from creating a deficit. Fiscal accounting also furnishes the necessary record against which the cost accounting should be reconciled and its correctness checked. In connection with the budgetary accounting it is usually required that the anticipated income be estimated, because construction programs must be planned in advance of receipt of the cash with which to pay for the construction. Budgetary control must extend over the award of contracts and incurrence of other obligations to be met out of anticipated income. Because the highway department income is largely from motor vehicle licenses and fuel tax receipts. it is not difficult to estimate rather accu-

rately the income for as much as a year or a year and a half in advance.

The fiscal accounting also deals with the debt service for outstanding obligations, usually highway bonds. The payment of interest and principal on bonds is usually a priority item after administration and maintenance expenses of the highway department are taken care of. The income budgeted for new construction is the residue after those three items are provided for.

Expenditures recorded under fiscal accounting are charged to expense or investment accounts. The former include all costs for general support of the department, primarily those for highway maintenance and overhead expenses in connection therewith. The expenditures for highway construction; land; building; road, shop, and laboratory equipment; and office furniture are charged to investment accounts, for the reason that the property purchased is used for a number of years before retirement. The original expenditure for these items reaches the expense or operation accounts through the annual depreciation charges.

Cost Accounting.—The accounting section is expected to determine the unit and total cost of the many operations performed by the several sections of the highway department. Unit operation costs for each kind of automotive equipment, cost of blueprinting and duplicating, cost of performing tests and analyses by the materials laboratory, percentage cost of engineering on construction projects, cost of operating stores and stocks, and cost of shop overhead are among unit operation costs determined by the accounting section. These unit costs are required in determining proper charges to other operations for which total costs are desired. For instance, in the maintenance operations, cost of the service of the use of motor trucks, graders, power shovels, and other heavy equipment is charged to road sections and work operations by the mile, hour, or day. The rates of charge—commonly called rental charges—are determined by analysis of the cost accounting records kept on each piece of equipment.

Compilation of Statistics on Costs.—The third function of the accounting department is to provide statistical information necessary and valuable to the various administrative officers in effecting economy of operation, in determining trends, and in furnishing the public information relative to the operation and

functions of the department. In this statistical field are data relating to the following: (1) mileage of the state highway system by types of surface, (2) investment in the state highway system by routes and by counties or other units, (3) incomes, (4) expenditures, (5) traffic, (6) motor vehicle registration, (7) trends of construction and maintenance costs, (8) trends in unit prices, (9) personnel and labor-hours utilized.

In carrying out its fiscal and cost accounting functions, the accounting section requires a rather long list of accounts. These include primary accounts, subaccounts, suspense accounts, memorandum accounts, and others. A general system of accounts, without reference to the detail and minor accounts, is described in subsequent paragraphs.

In addition to classification according to account, all expenses should further be classified as to objective—salary, hourly labor, travel expense, materials and supplies, equipment rentals, services, and the like as may be appropriate to each account.

Primary Accounts.—The six main groups of primary accounts used by many corporations and prescribed by certain regulatory commissions for use by public utilities are:

1. Fixed capital accounts.
2. Operating expense accounts.
3. Operating revenue accounts.
4. Income accounts.
5. Profit and loss accounts.
6. Balance sheet accounts.

These primary accounts furnish the material from which the following periodic statements are prepared:

1. Operating statement.
2. Income statement.
3. Profit and loss statement.
4. Balance sheet statement.

These primary accounts and periodic statements are those appropriate to highway department accountancy, modified, perhaps, with respect to revenue, income, and profit and loss accounts. A modification of these three account groups is probably desirable because highway departments differ in character from businesses operated with the expectation of earning a return. For that reason it seems desirable in highway depart-

ment accounting to omit the profit and loss accounts and to combine the revenue and income accounts. Moreover, the operating income, and profit and loss statement listed above may be combined into a single statement.

Each highway department, county or state, will need to arrange its accounts and classifications to meet the provisions of state laws, legislative requirements, administrative orders of the state highway commission, and federal aid regulations. The grouping of primary and subaccounts proposed on page 462 is perhaps the simplest arrangement that will be adequate for any highway department.

Fixed-capital Investment Accounts.—The fixed-capital investment accounts are designed to show the original cost and the depreciation to date for all properties devoted to the maintenance, operation, and administration of highways. To the accounts in this group should be charged the cost of highway construction, additions and betterments, replacements, value of right-of-way at time of acquisition, equipment, buildings, and cost of other property not fully consumed during the accounting year. Each account in this group should carry a credit account for the accrued depreciation.

Suitable property-record ledgers will be required for each item or group of similar items of property, on which the depreciations are credited as they accrue. For setting up these ledgers, the highways may be divided into unchangeable control sections, each with fixed geographical termini; these may be subdivided into subsections, of such length and characteristics that each may be accounted for as a unit. The fixed-capital accounts are usually designated by a number between 100 and 199, and a convenient subdivision is as follows:

Account 110—*Highways, Roads, and Streets.* This account should include the original costs of all highways, including surfacing and base, earthwork, structures, devices, appurtenances, engineering, and right-of-way. For purposes of administration and reports, the account should be subdivided further into major accounts for each road system, such as primary roads, secondary roads, municipal streets, and other classifications that are appropriate within the state. Furthermore, a subdivision common to each of these major subdivisions should show the items of the construction costs grouped conveniently for applying depreciation rates and probably maintenance requirements.

Account 120—*Equipment.* This account should include the cost of all equipment owned by the department with such subclassifications by types,

function, and organization as are desirable from an administrative and cost control standpoint.

Account 130—*Buildings*. This account should include the cost of all buildings segregated according to location and general purpose. Individual accounts should be kept for each building.

Account 140—*Land Other than Right-of-way*. This account should include the cost of land that is owned, there being a complete record for each parcel but with the parcels grouped according to use. Ordinarily, land is carried at the purchase price including legal and damage costs, if any, without depreciation or appreciation adjustments. In the event, however, that land values change materially from the purchase price, the accounts should be adjusted. It is advisable to depreciate annually over a comparatively short term those right-of-way costs of damages and other extraordinary costs above normal land values in order that the present book value of such land will more nearly approach normal land values. Quarries and mineral deposits should be written down each year in accordance with the depletion of the deposits. For these reasons, credit accounts for accrued depreciation or depletion are needed for all accounts for land.

Account 150—*Administration and Overhead Investment*. This account should include administration and overhead expenses that are chargeable to the construction of highways and acquisition of fixed property but which are not prorated to specific accounts or items. The total departmental expense of administration for a year should be prorated between fixed capital and operation expense, and the portion charged to fixed capital should be depreciated annually the same as physical property.

Account 160—*Suspense Investment Accounts*. This account should include the cost of properties that at the time may not be assigned to a final account. Particularly, it will be found convenient to charge to this account the monthly contractor's payments, pending settlement of the final estimates and allocation of the final cost to highways, roads, and streets.

Operating Expense Accounts.—The accounts devised under the title of operating expense are those required to show the expenses of furnishing the public with a highway transportation facility, including maintaining roadways, administration costs, and depreciation and maintenance of buildings and equipment. The depreciation of fixed-capital highway property is also a highway operating expense. These accounts are usually given members between 200 and 299, and a convenient subdivision is as follows:

Account 210—*General Administration Expense*. This account should include the cost of administrating the department, including salaries of administrative officials, wages, office expenses, office building expense, office equipment expense, and similar items of expense that are general in nature and cannot fairly be charged to a specific construction or operation account at the time of payment. All expense that can properly be charged

to specific expense accounts at time of payment should be so charged. The total cost of general administration is ultimately chargeable in part to fixed capital and in part to operation expense.

Account 220—*Rental, Proration, and Suspense Accounts.* The accounts suggested under this classification are those of service bureaus whose costs usually are eventually absorbed by other accounts and activities. Each account is finally closed out either to operating expense or to fixed property accounts. In most cases the expense would be closed out currently by use of purchase slips or service or rental charges. Each of the functions represented in these accounts is primarily one serving several other functions of the department and must be carried under separate account so that its cost may be determined and passed on to final accounts by the proper unit service or rental charge.

Account 230—*Highway Operation Expense.* This account should include the general operation expenses of the highway department, exclusive of those expenditures for fixed properties and for other functions not directly in connection with the furnishing of a complete highway facility.

Account 240—*Highway Depreciation.* This account should include the operation expense on account of depreciation of the highways and retirement of them. The classifications will follow those included under the corresponding fixed-capital accounts. The account should also include the depreciation of general administration, research, and other general property accounts that are not classified with highways, equipment, buildings, and other physical items.

Account 250—*Other Highway Department Expense.* This account should include separate subaccounts for those functions of the highway department which are not directly related to the furnishing and maintenance of the highway system and may be expense imposed by special state laws. It will be important to show the expense of collection of income when such duty is that of the highway department.

Operating Revenue and Income Accounts.—The factual situation as to the operation revenues and other income of a highway department differs in important particulars from the corresponding situation in the administration of other public utilities. State highway departments do not receive payments for services direct from those to whom services are rendered. Collections for road services rendered are made by various state, county, and federal agencies and go first into various public funds; they finally reach the highway departments by appropriation acts of state legislatures and of congress. Besides income from appropriations, highway departments may obtain funds by the sale of bonds. Since the bonds and bond interest are usually to be paid from future receipts, it often becomes necessary to forecast such receipts for 15 to 20 years ahead. When developing construction

programs and awarding highway construction contracts, anticipated future highway revenues are set up in some form of account against contractual obligations to show the true financial situation. In any case good management requires setting up regular budget accounts in advance of receipt of anticipated revenues. Provision is made in the balance sheet Account 400 for anticipated revenues and expenditures, the latter being primarily contractual and budgetary obligations to be met out of future income.

The revenue and income accounts (the 300 group) provide only for crediting receipts and debiting expenditures for fixed-capital investments, operating expenses, and payments of interest and principal on debts.

General Balance Sheet Accounts.—Highway accountants are called upon to prepare "general balance sheet" statements, each giving the status of the highway system at a particular date. The accounts needed for such balance sheets are numbered from 400 to 499. These balance sheet accounts are listed in two columns, corresponding to the asset (the left hand) and liability (right hand) sides of the conventional balance sheet form.

Account 411 provides for cash and cash receivable and for prepaid expenses.

Account 412 provides for the office, engineering, laboratory, and stores and shop inventories assets.

Account 413 shows the contingent current assets. It is here that estimated future incomes, sometimes not receivable for one or more years, come into the highway accounts.

Account 414 is for sinking funds or bond retirement funds, if any.

Account 415 shows the fixed-capital assets, which may amount to several hundreds of millions of dollars. The main asset items are the actual highways, the road equipment, and the highway shops and buildings of various kinds.

It is usually customary to carry these fixed assets at actual cost new on the asset side of the balance sheet, with a corresponding liability account "Reserve for Depreciation" on the liability side. The form in which the physical assets are carried at their present depreciated values, instead of their values new, the depreciation reserve account disappearing from the liability side, is perhaps preferable for highway departments.

Account 421 includes the current liabilities, on the right-hand side, such as accounts payable, wages, and interest payable.

Account 422 shows the budgetary liabilities. Like the contingent current asset accounts, these deal with estimated future transactions (in this case estimated future expenditures).

Account 423 shows the deferred liabilities such as advance deposits, reserves for employees' insurance and retirement, and other liabilities for the future.

The form of balance sheet suggested is the dual balance sheet. Subtracting the sum of current liabilities, budgetary liabilities, and deferred liabilities from the sum of current assets, inventories, and contingent current assets gives the current surplus. Subtracting the funded debt liability from the sum of sinking funds and fixed-capital assets gives the net capital surplus.

List of Standard Accounts.—The following list of accounts is suggested as representing the arrangement suitable for state highway department accounting. Corresponding accrued depreciation accounts for each fixed-capital investment account are omitted in this listing but are necessary to complete the account groups.

100. Fixed-capital Investment Accounts

110. Highways, roads, and streets:
    111. Right-of-way.
    112. Roadway drainage, grading, and earthwork.
    113. Drainage structures and earthwork protective structures.
    114. Roadway surfacing (by type).
    115. Roadway base and foundations.
    116. Improved shoulder and approach surfacing.
    117. Bridges, viaducts, grade separations, tunnels, and other structures (by individual structure).
    118. Traffic and pedestrian services.
    119. Roadside development.
120. Equipment:
    121. Road equipment.
    122. Shop and service equipment.
    123. Office and administrative equipment, furniture, and files.
    124. Engineering and laboratory equipment.
130. Buildings:
    131. Central office buildings.
    132. Central shop and warehouse buildings.
    133. Field office buildings.
    134. Field shop and warehouse buildings.
140. Land other than right-of-way:
    141. Central building land.
    142. Field building land.
    143. Gravel deposit land.
    144. Rock quarry land.
    145. Other land.
150. Administration and overhead investment:
    151. General administration.
    152. Research and planning.

160. Suspense investment accounts:
 161. Earnings on work under construction.
 162. Loss of property due to catastrophe.

200. Operating Expense Accounts

210. General administration expense:
 211. Commission and state expense.
 212. General office expense.
 213. General accounting office expense.
 214. Legal expense (other than right-of-way).
 215. Office building expense.
 216. Other administrative general expense.
220. Rental, proration, and suspense accounts:
 (These service accounts are all to be closed out to other accounts.)
 221. Stores and stocks.
 222. Material deposits.
 223. Blueprinting and duplicating.
 224. Shop overhead.
 225. Equipment operation.
 226. Engineering (office and field).
 227. Testing laboratory.
 228. Research and planning.
230. Highway operation expense:
 231. Highway maintenance (by highway section):
  231.1 Routine roadway surface operations.
  231.2 Special roadway surface operations.
  231.3 Shoulders and approaches.
  231.4 Roadside and drainage.
  231.5 Traffic services.
  231.6 Snow, ice, and sand control.
  231.7 Bridges, viaducts, and tunnels.
  231.8 Extraordinary repair and maintenance due to catastrophe.
  231.9 Maintenance general expense.
 232. Operation of drawbridge, toll and ferry facilities (by each facility). Structure maintenance under highway maintenance Account 231.
240. Highway depreciation:
 241. Depreciation of highways and facilities.
 242. Transfer of highways to other jurisdictions.
 243. Other depreciation expense not charged to specific accounts.
250. Other highway department expense:
 251. Revenue collection expense (by each source).
 252. Highway patrol.
 253. Tourist bureau and publicity.
 254. Outdoor advertising regulation.
 255. Testing and engineering expense for others.
 256. Radio operation.
 257. Other department expense.

300. Operating Revenue and Income Accounts

310. Debits:
    311. Fixed-capital investment expenditures.
    312. Operating expense accounts.
    313. Interest payments on debt.
    314. Extinguishment of debt principal.

320. Credits:
    321. General highway user revenue.
    322. Specific highway user revenue.
    323. U.S. Government aid.
    324. Legislative appropriations from general funds.
    325. Miscellaneous income.
    326. Receipts other than cash.
    327. Income from borrowing.

400. General Balance Sheet Accounts

410. Asset and other debits:
    411. Current assets.
    412. Inventories.
    413. Contingent current assets.

    Total current assets ________

    414. Sinking funds.
    415. Fixed-capital assets (less reserve for depreciation.

    Total assets ________

420. Liabilities and other credits:
    421. Current liabilities.
    422. Budgetary liabilities.
    423. Deferred liabilities.
    424. Current surplus.

    Total current liabilities ________

    425. Funded debt.
    426. Net capital surplus.

    Total liabilities ________

CHAPTER XIX

SELECTION OF TYPE OF ROADWAY SURFACE

The selection of the type of roadway surface is largely a matter of economic analysis for which only few fundamental data have as yet been established. The selection should be based on the principle of securing transportation at the minimum cost and no effort should be spared to find out what type of roadway surface will accomplish that in any community.

It certainly is not at present possible to determine transportation cost with absolute certainty in every administrative area and perhaps it will never be possible to do so. It is unlikely that the need for the exercise of good engineering judgment and thorough analysis of much technical data can ever be eliminated from the all-important problem of selecting the type of roadway surface.

Nor is it likely that the effect of the commercial promotion of paving materials can be wholly ignored. Since the administration of street improvements is largely political and that of rural highways is becoming more so every year, the engineer must reckon with political influences in the selection of types of roadway surface.

This presentation seeks to outline methods of selection that are perhaps more ideal than practical and are therefore inapplicable in many instances. On the other hand, the consideration of an ideal method may assist those who must perforce depart therefrom, to avoid the more serious breaches of good engineering.

Cost and Price Considerations.—The first step in the selection of the type of roadway surface, should be to determine, on the basis of current prices, the type that will give transportation service at lowest unit cost. If such a study results in any one type showing a marked superiority to the others considered, that is fairly positive evidence. A different type should be selected in such cases only under compulsion.

The cost comparisons may show that relative prices in a locality are such that several types of roadway surface will furnish

transportation at about the same cost. If that happens, the selection may be finally determined on the basis of physical characteristics of the pavement or from expediency.

Physical Characteristics of Roadway Surfaces.—Certain physical characteristics of roadway surfaces enter into the cost of transportation on that surface. Some such characteristics are, rolling resistance, rate of tire wear, smoothness, resiliency and rate and manner of wearing. The significance of these characteristics is included in proper cost comparisons.

Other physical characteristics are only partly covered or entirely excluded from any possible cost comparisons. These are principally of importance because of their reaction upon passengers in automobiles and busses.

Color.—The color of the roadway surface has some relation to the comfort of motor vehicle drivers. A type that glares or is harsh to the eye is somewhat less desirable than one that has a dead color and does not reflect light. This is perhaps a small factor.

Of more importance in high-class business or residence districts is the property of harmonizing with the surroundings. The black top pavements seem to be superior to the other types in this respect. In the locations mentioned the engineer would be justified in paying something for this characteristic.

General Appearance.—The general appearance of the pavement is dependent upon the physical makeup of the surface. Block pavements that are filled with a material of a color differing from that of the blocks, or a filler that wears off in irregular patches, do not have as pleasing an appearance as those of uniform color. The black, irregular streaks on concrete pavements that result from maintenance operations have been the occasion for much unfavorable comment, and illustrate the point. This consideration is of little moment in connection with the selection of surfaces for rural highways but would be of some importance on certain city streets.

Dustlessness.—The effect of the dust from earth, gravel or macadam surfaces upon the vehicle should be reflected in vehicle costs since the dust contributes materially to the cost of vehicle maintenance. But the effect of dust upon travelers is not reflected in any possible cost estimate. There is a distinct value to absence of dust, from the standpoint of the comfort of the traveler, although it would be exceedingly difficult to place a

money value on it. Under certain conditions the dust hangs over the roads in such clouds as to make driving exceedingly unpleasant and even dangerous. The traveler would certainly be willing to pay a cent a mile and perhaps more to be relieved of the dust. At any rate dust is inherent in some types of roadway surfaces a fact that must be taken into account in the selection of types of improvement.

Ease of Cleaning.—The cost of cleaning is properly a part of the cost of pavement maintenance, although it is seldom so reported. It should be included in the cost items upon which relative costs are based; and if that is done, no other account need be taken of the relative ease with which the several types can be kept clean and sanitary. To a limited extent the question of sanitation is involved in the selection of type of roadway surface. In the vicinity of markets and of wholesale warehouses where perishables are handled and in some dock districts considerable decayed matter will be scattered about on the pavements. If there are cracks or joints into which such matter can be packed, an ill-smelling and unsanitary condition results which is highly objectionable. In such locations some account must be taken of the ease with which certain roadway surfaces may be kept sanitary.

Safety.—The coefficient of friction of a roadway surface is a measure of the contribution of the surface toward the safety of travel thereon. The high-type pavements can be constructed with an ample coefficient of friction by employing the proper materials and compounding them correctly. Several of the low-cost types require, or may be built with, an excess of binder and in consequence have a tendency to a low coefficient of friction (Table XVII) in very hot weather or when wet or frosty. The wood-block and stone-block pavements have become obsolete because among other objectionable characteristics they had a very low coefficient of friction when wet.

SPECIAL LOCAL CONSIDERATIONS

In a few instances the engineer will be confronted with the problem of selecting the type of roadway surface in the face of local conditions of a special nature or those urged upon him as special. The problem is not difficult in such cases if the cost (not price) is adversely affected; but when that is not the

case or when little difference exists in cost, some embarrassing situations arise.

Local Materials.—If local supplies of road material are commercially available, that fact is likely to be reflected in the prices of the various types of roadway surfaces. When cost comparisons have been made, the influence of local materials has already been included, but sentimental reasons are sometimes urged in favor of local enterprise. Properly so in many cases. Undeveloped local supplies of material do not appear as a factor in any cost analysis unless effort is made to have them considered and a basis of development arranged. A contractor cannot base his bid on the use of local gravel or other material, unless he knows that the material is of acceptable quality and will be made available in ample quantities at a prearranged price. It is the task of the engineer to ferret out the local materials and make sure that the cost analysis takes account of them.

Existing Types.—There is some advantage in having a single type of roadway surface in an administrative unit but the probability is that when it becomes generally known that a city or a state favors this or that type, the price thereof will gradually increase. The best policy to pursue seems to be to accept either of two types of roadway surface of such character that they are inevitably always in competition. The maintenance organization and equipment is simplified somewhat by confining the types to a few classes. No sacrifice in cost is warranted in order to keep a system of highways uniform in type, but where no appreciable difference in cost exists between several types, it is advantageous to standardize on two of them.

Maintenance Organization.—All of the states have developed, or are now developing, competent maintenance organizations and can maintain any of the types of roadway surface. A good many counties and cities also have competent maintenance organizations but on the other hand the great majority of the smaller cities, the townships and the counties seem unable to accomplish timely and proper maintenance. There is no type of roadway surface that will give maximum serviceability when maintenance is neglected, but some types withstand abuse better than others. This fact needs to be taken into account frequently when types of roadway surface are being selected.

Recapitulation.—The selection of the type of roadway surface is primarily a problem of estimating which available and accept-

able type will furnish transportation service at the lowest cost. Where there is a markedly lower cost for any particular type it should generally be selected.

In arriving at the estimated cost of transportation there should be consideration of every type of roadway surface that is suitable for the location.

In rather frequent instances it will be found that the cost of transportation will be nearly identical for several types of roadway surface. In such instances the selection may be based on considerations other than cost.

A few characteristics of certain types of roadway surfaces are not reflected in the cost of transportation on those types. The engineer must arbitrarily evaluate such characteristics if one of these types is economically desirable.

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